water affairs
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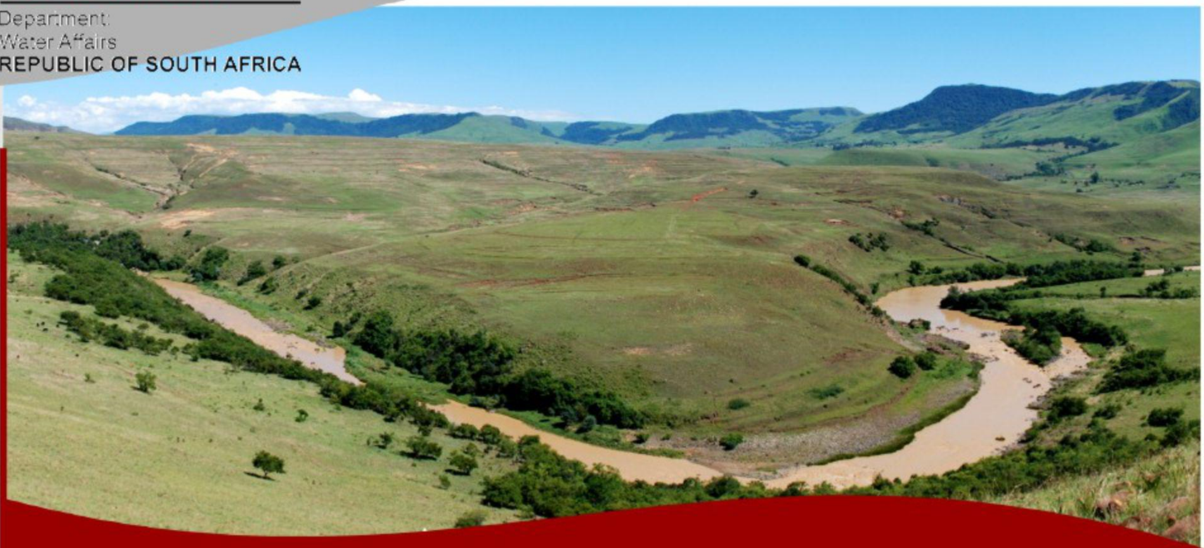

REPORT NO: P WMA 11/U10/00/3312/3/1

The uMkhomazi Water Project Phase 1: Module 1: Technical Feasibility Study: Raw Water

ENGINEERING FEASIBILITY DESIGN REPORT

VOLUME 1

FINAL

MAY 2015

W100_2013/J01763

Project name:	***The uMkhomazi Water Project Phase 1: Module 1: Technical Feasibility Study Raw Water***
Report Title:	***Feasibility Design Report***
Sub-report title:	***Volume 1***
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CONSULTANTS: AECOM (BKS*) in association with AGES, MM&A and Urban-Econ.

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Mogoba Maphuthi and Associates — Mogoba Maphuthi & Associates (MMA)

Urban-Econ — UE Urban-Econ Development Economists

PREAMBLE

In June 2014, two years after the commencement of the uMkhomazi Water Project Phase 1 Feasibility Study, a new Department of Water and Sanitation was formed by Cabinet, including the formerly known Department of Water Affairs.

In order to maintain consistent reporting, all reports emanating from Module 1 of the study will be published under the Department of Water Affairs name.

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The uMkhomazi Water Project Feasibility Design Report is divided into two volumes.

The first volume (**Volume 1**) contains the main text, dealing with the technical aspects as well as the financial aspects of the project.

The second volume (**Volume 2: Annexures**) contains the annexures to the *Feasibility Design Report (P WMA 11/U10/00/3312/3/1)* which are referred to in the text and is numbered according to the chapters in **Volume 1**. The annexures contain figures, tables and information which, for ease of reading, have been removed from **Volume 1**.

Volume 2 is divided into the following annexures:

- Annexure 3 – Smithfield Dam
- Annexure 4 – uMkhomazi – uMlaza Tunnel
- Annexure 5: Langa Dam
- Annexure 6: Raw Water Pipeline
- Annexure 7: Hydropower Plant
- Annexure 8: Flow Gauging Weirs
- Annexure 9: Roads
- Annexure 10: Waste Disposal Sites
- Annexure 11: Land Acquisition
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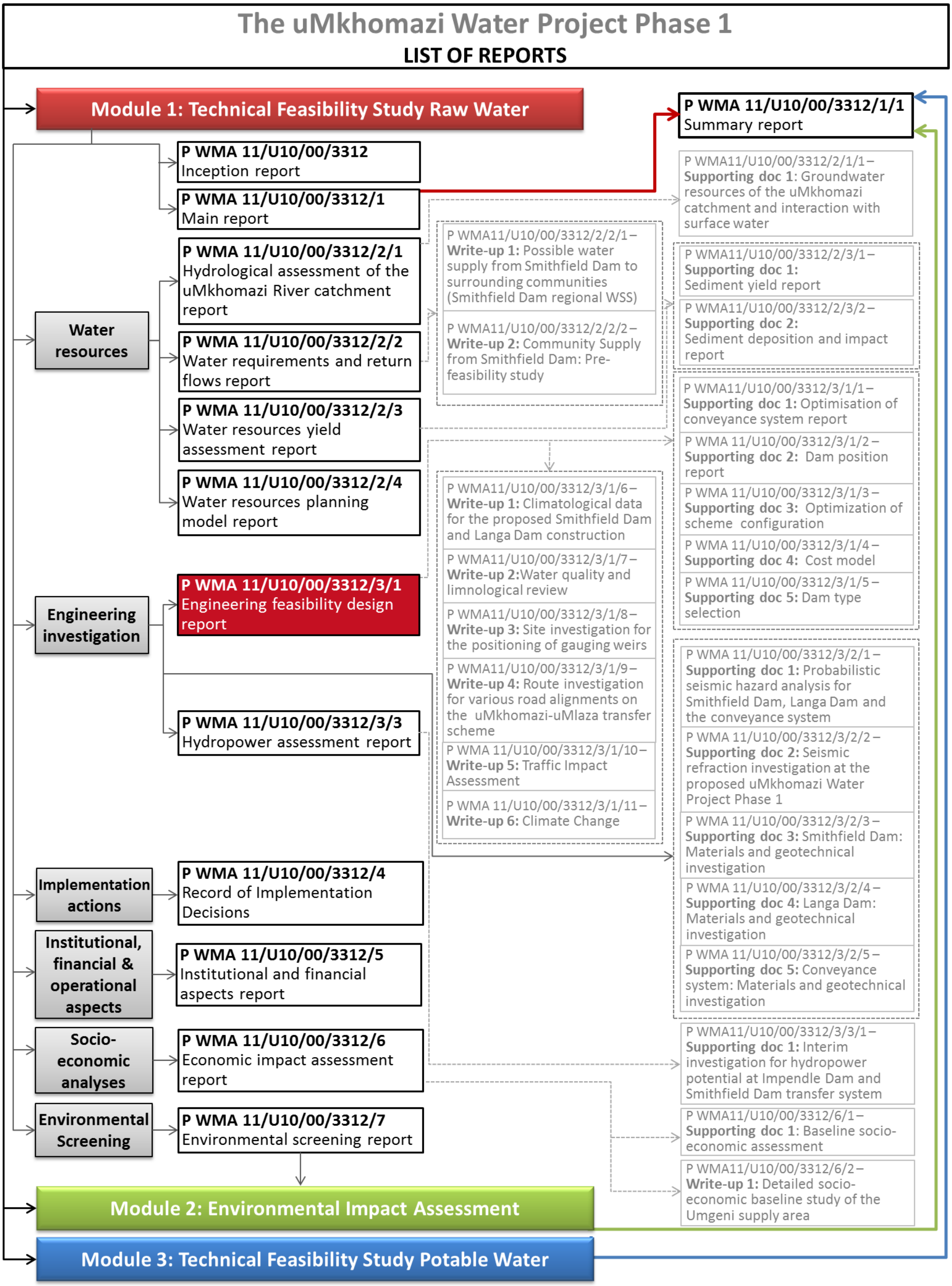
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Executive summary

This report covers the feasibility design of the raw water component of the uMkhomazi Water Project (uMWP), a water transfer scheme from the uMkhomazi River to the uMngeni River Catchment, which is required to supply water as from 2023 to the eastern part of the Umgeni Water (UW) Supply area.

The pre-feasibility study recommended a dam in the uMkhomazi River at the Smithfield Dam site and a conveyance system from there to the Umlaas Road pipeline of UW.

*This scheme was analysed and optimised until a final scheme, including a pressure conveyance system, was selected. This scheme consists of the following raw water infrastructure components described in **Table i**.*

Table i: uMWP raw water infrastructure components of the selected scheme

Major component	Subcomponent
Smithfield Dam	Smithfield Dam Flow Gauging Weir (s) Smithfield Tunnel Intake Tower Impendle Flow Gauging Weir Access Roads Deviation of public roads Deviation of Transmission Lines Smithfield Dam Hydropower plant
uMkhomazi – uMlaza Tunnel (pressure system)	Smithfield Waste Disposal Site Tunnel Access Adits Access Roads Ventilation Shafts Central Waste Disposal Site
Baynesfield Raw Water Pipeline (pressure system)	Baynesfield Hydropower Plant Access Roads
Langa Dam	Access Roads

The feasibility study is described in several reports as shown in the list of reports of the feasibility study on the previous page. These reports include, inter alia, descriptions of the following:

- *Geotechnical investigations, including construction materials and foundations for structures;*
- *Water requirements;*
- *Water yield analyses including in-stream flow analyses;*
- *Smithfield Dam position;*
- *Optimisation of conveyance system; and*
- *Optimisation of scheme configuration.*

Furthermore, topographical investigations were carried out and all the information obtained was used as base information for the feasibility design.

SMITHFIELD DAM

The size of this dam, situated on the uMkhomazi River at the farm Smithfield, was optimized in terms of cost and as a result a 31% MAR dam size was selected. Further dam type selection studies, focussed on making maximum use of the available construction materials, showed that the following type and layout is the lowest in cost and met construction programme requirements:

- *An earth core rockfill dam (Main Dam) constructed with residual dolerite earthfill core and dolerite rockfill in the outer zones;*
- *A primary side channel spillway with a gravity weir structure, chute and ski jump structure;*
- *A secondary fuse plug spillway;*
- *A permanent bottom outlet laid out as an intake structure to one of the two 8 m diameter tunnels (used initially for river diversion tunnels) with an access bridge from the main dam crest;*
- *A zoned earthfill embankment Saddle Dam; and*
- *An intake tower and access bridge in the Smithfield Dam Reservoir to the transfer tunnel.*

The feasibility design of the dam included low frequency and high frequency flood estimates, flood attenuation studies, identification of the required freeboard and associated crest height, consideration of the possible flood surges from landslide induced waves from the reservoir and effect on freeboard, summary of the geotechnical investigations for foundation and construction materials, river diversion, embankment dam zoning and slope stability analyses, the design consideration of various spillways including a side channel spillway as well as a fuse plug spillway and the design of a multi-draw-off double pipe system dam outlet. Provision is made at this tunnel inlet also for the release of water to the second tunnel for when the second phase of the uMWP is implemented. The design of this dam is described in detail in ***Section 3*** *of this report.*

A detailed river diversion method to construct the dam, including 6 cofferdams and two 8 m diameter tunnels, is described in this report.

During tender design the following must be carried out:

- *The excavation for the founding level of the main embankment will yield a large volume of material, which might be suitable as impervious and semi-pervious earthfill for the Saddle Dam Embankment. Sampling and laboratory testing of this material will have to be conducted to confirm the suitability.*
- *Testing for a grout curtain for the saddle embankment is recommended to a level at least 20 m below the quarry floor due to the development of Quarry I, just upstream of the embankment of the Saddle Dam. Although grout penetration might be very small, the drilling, water testing and grout take from a grouting operation is very important and must be considered in the final stage of a geotechnical investigation when sub-surface information is obtained at close intervals below the footprint of the dam.*
- *Additional geotechnical investigations are required to determine foundation conditions for the position of the main spillway as well as the erosion potential at the foundation and downstream area of the fuse plug spillway. Subsequently, a total cost optimisation of the dam freeboard, Main Spillway and spillway chute width should be carried out. This may result in discarding the fuse plug spillway and having one large main side channel spillway.*
- *Hydraulic model testing of the following components should be carried out:*
- *Side channel spillway as well as fuse plug spillway if erosion potential downstream of the fuse plug spillway is acceptable.*
- *Intake Tower to the Tunnel to test air entrainment and acceptable hydraulic conditions.*
- *The spillway and freeboard arrangement should be optimized after the geotechnical investigations and the hydraulic model study have been completed. Anchors, the drainage system as well as the slope protection in the form of shotcrete for the spillway chute should be designed.*

UMKHOMAZI – UMLAZA TRANSFER TUNNEL

The transfer tunnel extends from the left side of the Smithfield Dam reservoir to the upper reaches of the Mbangweni Dam in the Mbangweni River. The shortest route through the mountain range between the two valleys was identified based on a comparison analysis between pumping schemes and the selected gravity conveyance system. The tunnel is 32.5 km long and the selected optimum inside diameter for a discharge at peak demand of 8.65 m³/s is 3.5 m diameter. This pressure tunnel has to be driven through hard quality shales and dolerites (last mentioned about 40% of the distance) and is connected with a pressure pipeline from the tunnel outlet to the site of the Baynesfield Water Treatment Works (WTW). This system is sized to accommodate *design flows with Smithfield Dam at the minimum operating level (MOL). The design of this component is described in detail in **Section 4** of this report.*

Based on lower cost and limiting the critical construction path of the project the tunnel has been designed for two Tunnel Boring Machines (TBMs). The tunnel was laid out with a slope in an upward direction to the west and the TBMs have to bore in the upstream direction (west), accommodating encountered ground water in gravity flow requirements. The tunnel is designed for drainage during construction as seepage from groundwater is expected. Some parts of the tunnel, including access adits, are to be excavated using drill and blast techniques.

To ensure stable rock portals the inlet and outlet portals of the tunnel are to be formed through excavation of weathered rock materials. These excavated materials are available for construction purposes. An intake tower to the tunnel is described as part of Smithfield Dam.

Air entrainment of the tunnels to accommodate flowing water will be facilitated through two ventilation shafts, a central access adit and a shaft and pipe through the intake structure.

The following recommendations are made for the tunnel design phase:

- *Further geotechnical investigations to assess geotechnical and groundwater (including quality) conditions as well as the lining requirement of the tunnel should be carried out during the tender design stage; and*

For this phase a complete concrete lining system has been adopted.

LANGA DAM

*The Langa Dam site is located in the Mbangweni River at the downstream end of the tunnel outlet portal on the farm Baynesfield. This dam is required for storing water and for supply to UW during emergency and maintenance of the 32.5 km long tunnel. It is connected to the Baynesfield Raw Water Pipeline. When full at 923 masl, it has a 24-day supply storage at the maximum supply rate of the conveyance system. The Mgeni Water Supply System (WSS) will, with the Langa Dam, provide for the required two months' during emergencies and maintenance periods of the tunnel when no water will be supplied through the tunnel. The dam is described in detail in **Section 5** of this report.*

A selection study has indicated a Concrete Face Rockfill Dam to be the most feasible dam type for construction. The rockfill will consist of shales from the reservoir (dam *basin) of the dam. Finer bored rockfill from the tunnel will be used on its downstream toe section and the rest of the excavations from the outlet portal will be stored in a berm on the downstream side of the dam. A dolerite rockfill layer will be used as protection of the shale rockfill where exposed.*

Furthermore, a side spillway on the left flank will ensure that the dam cannot be overtopped during flood events or filling from Smithfield Dam. A double one level draw-off pipe intake system in an intake tower and bottom outlet would facilitate water to fill the dam from Smithfield Dam under gravitation, as well as to make releases in support of downstream ecological water requirements and water supply when required.

In order to achieve the maximum overall system yield, accommodate the relevant requirements and maintain the water volume in Langa Dam at acceptable levels (i.e. as full as possible) the following operation rule is recommended:

- *Langa Dam should initially be filled from Smithfield Dam via the conveyance system (tunnel and pipeline).*
- *Subsequently, when required Langa Dam should be refilled (i.e. kept at FSL) from Smithfield Dam. However, support should only be provided when Smithfield Dam is spilling and all downstream ecological water requirements in the uMkhomazi River have been fully supplied.*

The following recommendations are made:

- *Further geotechnical investigations are required during tender design regarding the bottom outlet.*

BAYNESFIELD RAW WATER PIPELINE

*A pressure pipeline will connect the Tunnel outlet to Langa Dam and the Baynesfield WTW, and is described in **Section 6** of this report. The main section of this pipeline has an internal diameter of 2.6 m and connects the Tunnel with the WTW. A connection pipeline of 1.6 m diameter will also be provided to the Langa Dam. It will be laid out to accommodate the wetlands and to follow a route which facilitates the energy line. Bedding materials for laying the pipeline should be imported from processed tunnel muck from the tunnel or commercial sources.*

A stilling basin will be provided at the end of the pipeline for dissipating the energy of the water before it is routed through the WTW.

HYDROPOWER PLANTS

*As part of the scheme, hydropower development was considered wherever water flow and head warranted. Two sites have been identified for hydropower generation as discussed in **Table ii**.*

Table ii: Hydropower development information

Name and Location of Hydropower Plant (HPP)	Buyer of electricity	Power Potential (MW)
Baynesfield HPP: At end of Pressure Tunnel and Pipeline Conveyance Structure	*The WTW Owner (probably UW) with supply from the national Grid as backup.*	3.0
Smithfield Dam HPP: At Smithfield Dam	*ESKOM national grid for operation and maintenance of Smithfield Dam*	2.6

*The preliminary design of the possible hydropower plants as well as the economic sustainability is described in this report in **Section 7**. The cost estimates are included in the summary of total costs of the scheme.*

Based on the assessment of the economic sustainability of these options, it was found that the wheeling of power into the grid is a feasible option for both Baynesfield HPP and Smithfield Dam HPP. For the latter, high hydropower generation is needed for economic feasibility. It is recommended that these possibilities be discussed with Umgeni Water, to determine whether they would be interested in such an arrangement, and that the arrangements with Eskom are confirmed. The development through Public Partnerships and or Small Medium Enterprises (SME) can be considered.

An option, which may show economic feasibility with further investigations, is the use of power generated at Smithfield Dam HPP to directly supply local facilities needed to operate and maintain Smithfield Dam. It is recommended that a detailed cost assessment of the civil, hydro-mechanical and power transmission components be undertaken, to determine whether this small scale hydropower scheme would be feasible. This would allow for the dam to be operated independently of the grid. This arrangement would need to be confirmed with Eskom.

Further investigations should also be done to determine parties that would be interested with linking the scheme to a renewable energy program for small hydropower schemes, to determine the potential cost benefits of this.

The scheme has to be designed to accommodate hydropower development.

FLOW GAUGING STATIONS

Three flow gauging weirs are required to measure river flows at the following positions and for indicated reasons:

- *Weir 1: Upstream of Impendle Dam to measure inflow to Smithfield Dam;*
- *Weir 2: Downstream of Smithfield Dam to determine the lower portion of discharges from Smithfield Dam and to monitor in-stream flow requirements; and*
- *Weir 3: Near EWR/IFR2, further downstream of Smithfield Dam. This will determine the run-off from the incremental catchment downstream of Smithfield Dam to assist with determining and monitoring the ecological water requirement.*

*The design of these weirs as crump weirs accommodating site specific flows is described in **Section 8** of this report.*

Geotechnical investigations for these weirs should be carried out during the tender design stage.

ACCESS AND DEVIATION OF ROADS

*The following roads were identified, for which route determination is addressed in this report (**Section 9**):*

- *Smithfield Dam*
    - *Deviation of the R617*
    - *Access road to Nonguqa*
    - *Access road to tunnel inlet portal*
    - *Access road to dam wall*
    - *Construction road*
    - *Main access road*
- *Tunnel*
    - *Access road to Ventilation Shaft 1*
    - *Access road to Ventilation Shaft 3*
    - *Access road to centre adit entry*

- *Langa Dam*
  - *Access road to tunnel outlet portal and Langa Dam (Option 1)*
  - *Access road to tunnel outlet portal and Langa Dam (Option 2)*
  - *Access road to WTW*
- *Gauging weirs*
  - *Access roads to gauging weir 1*
  - *Access road to gauging weir 2*
  - *Access road to gauging weir 3*

No detail geotechnical investigations as well as construction materials were carried out on the routes. These investigations have to be carried out during the tender design phase.

The deviation of road R617 is a major project and the designs have to be approved by the Roads Authorities.

WASTE DISPOSAL SITES

Three waste disposal sites have been identified for disposal of construction materials during the construction of the uMWP1 and will form part of the EIA application. However, only two waste disposal sites, one near the tunnel inlet portal and one midway along the tunnel length near the central tunnel access adits, will be used. These have been discussed in detail in ***Section 10****.*

Excavated material from the uMkhomazi – uMlaza Tunnel and the portals will be mainly disposed of at these sites. Tunnel muck and excavated material from the downstream outlet portal will be used for the construction of Langa Dam and thus the development of the third waste disposal site is not necessary.

The waste disposal sites will only be operational for the construction period of uMWP1 and will be rehabilitated afterwards.

The new National Norms and Standards for the Disposal of Waste to Landfill (DEA, 2013) classifies the sites as Class D landfills with Type 4 waste (building and demolition waste and excavated earth material not containing hazardous waste of hazardous chemicals).

COST ESTIMATE

*Detailed cost estimates of all construction activities were undertaken for all components of the uMWP1, comprising quantities and rates. A summary of the total scheme cost estimate for the raw water system is shown in **Table iii**. Further detail on the cost estimate and methodology followed is given in **Section 14**.*

Table iii: Summary of total cost estimate for the raw water system

Component	*Cost (R million, excl. VAT)*
Smithfield Dam	*2 018*
uMkhomazi – uMlaza tunnel	*3 901*
Langa Dam	*439*
Baynesfield Raw Water Pipeline	*365*
Transmission lines	*5*
Smithfield Dam and Baynesfield hydropower plants	*83*
Waste disposal sites	*15*
Flow gauging weirs	*30*
Roads and bridges	*232*
Sub-total of activities	***7 088***
P&G costs (25% of activity cost)	*1 772*
Professional fees (12% of activity cost)	*851*
Environmental, landscaping and social costs (lump sum)	*450*
Land acquisition (lump sum)	*37*
Sub-total of activities and value-related costs	***10 198***
Contingencies (25% of above sub-total)	*2 550*
Implementing agent - TCTA (lump sum)	*200*
Total: Raw water system	***12 948***

*A summary of the project cost estimate as per envisaged tender package is included in **Table iv**.*

Table iv: Project cost estimate summary per envisaged tender package

Component	*Cost (R million, excl. VAT)*
Smithfield Dam	
Smithfield Dam	*2 018*
Access and deviation of roads	*187*
Flow gauging weirs	*30*
Waste disposal site 1	*7*
Transmission lines	*5*
Smithfield Dam HPP	*38*
Sub-total: Smithfield Dam	***2 288***

Component	*Cost (R million, excl. VAT)*
uMkhomazi – uMlaza tunnel	
uMkhomazi – uMlaza tunnel	*3 901*
Access and deviation of roads	*12*
Waste disposal site 2	*7*
Baynesfield HPP	*45*
Sub-total: uMkhomazi – uMlaza tunnel	***3 966***
Langa Dam	
Langa Dam	*439*
Baynesfield Raw Water Pipeline	*365*
Access and deviation of roads	*30*
Sub-total: Langa Dam	***834***
Sub-total of activities	***7 088***
P&G costs (25% of activity cost)	*1 772*
Professional fees (12% of activity cost)	*851*
Environmental, landscaping and social costs (lump sum)	*450*
Land acquisition (lump sum)	*37*
Sub-total of activities and value-related costs	***10 198***
Contingencies (25% of above sub-total)	*2 550*
Implementing agent - TCTA	*200*
Total: Raw water system	***12 948***

*A cash flow forecast was also done to estimate the annual expenditure over the construction period. This detail is contained in **Section 14.5**.*

CONSTRUCTION PROGRAMME

*A construction programme for this comprehensive multidisciplinary project, including the principal work items, with emphasis on the critical path activities, is discussed in **Section 15**. The construction programme has been determined based on the following milestone dates:*

- *Commencement of construction: September 2018.*
- *Commencement of Impoundment: September 2022.*
- *Commencement of water supply to UW: January 2023.*
- *Determining of construction activities based on realistic construction production rates.*

With the programme it was assumed that appropriate time has been allocated to complete pre-construction activities and preparations. These include:

- *Tendering process and contract award;*
- *Obtaining of relevant approvals, permits and licenses;*
- *Financing; and*
- *Land acquisition.*

From the construction programme it is clear that the critical path follows the uMkhomazi – uMlaza Tunnel construction preparations and activities. These include:

- *Mobilisation of the tunnel boring machines;*
- *River crossing;*
- *Erection of the crusher and batching plant;*
- *Drilling and blasting of the central access tunnel at mid length of the tunnel;*
- *Drilling and blasting of other access adits; and*
- *Boring of uMkhomazi – uMlaza Tunnel from chainage 33.1 km to chainage 15.2 km*

All the other facilities can be completed within this critical period. However, the completion of other construction activities is also crucial even though it is not on the critical path. These activities include:

- *Construction of access roads;*
- *Excavation and lining of the Smithfield Dam River Diversion Tunnel 2;*
- *Construction of Smithfield Dam RCC Cofferdam 5;*
- *Construction of Smithfield Dam Rockfill Cofferdam 6; and*
- *Construction of a large portion of the Smithfield Dam Saddle Dam.*

To adhere to the milestone dates, it is of utmost importance to stay on track of the construction programme, especially the activities associated with the critical path as well as the other activities deemed important. It is recommended that all must be done to commence construction in 2017 to ensure that delays can be mitigated.

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LIST OF ABBREVIATIONS

ACRU	Agricultural catchments runoff units model
BCM	Bank cubic metre

BoQ	Bill of Quantities
CFR	Concrete faced rockfill
CVC	Conventional vibrated concrete
DBT	Drill and blast techniques
DWA	*legacy* Department of Water Affairs
DWS	Department of Water and Sanitation
ECR	Earth core rockfill
EIA	Environmental Impact Assessment
ESL	Emergency supply level
EWR	Ecological water requirement
FoS	Factor of safety
FSL	Full supply level
FSC	Full supply capacity
HFY	Historic firm yield
HPP	Hydropower plant
ICOLD	International Commission on Large Dams
IVRCC	Immersion-vibrated roller compacted concrete
LCM	Loose cubic metre
LDL	Lowest drawdown level
MAP	Mean annual precipitation
MAR	Mean annual runoff
Masl	Metres above sea level
MOL	Minimum operating level
NGL	Natural ground level
NOC	Non-overspill crest
NPV	Net present value
O&M	Operation and maintenance
OCS	Off-channel storage
OHSA	Occupational Health and Safety Act
P&G	Preliminary & general
PMF	Probable maximum flood
PSP	Professional Service Provider
RCC	Roller compacted concrete
RDF	Recommended design flood
RID	Record of implementation decisions
RMF	Regional maximum flood
RWSS	Regional Water Supply Scheme
SANCOLD	South African National Committee on Large Dams
SEF	Safety evaluation flood
SUH	Synthetic unit hydrograph
TBM	Tunnel boring machine

TOR	Terms of Reference
uMWP	uMkhomazi Water Project
URV	Unit reference value
USBR	United States Department of the Interior, Bureau of Reclamation
VAPS	Vaal Augmentation Planning Study
VAT	Value added tax (14%)
WRC	Water Research Commission
WRPM	Water Resources Planning Model
WRYM	Water Resources Yield Model
WTW	Water treatment works

LIST OF UNITS

Ha	Hectare
km^2	square kilometre
m	Metre
m^3	cubic metres
m^3/s	cubic metres per second
masl	metres above sea level
R	Rand
$t/km^2.a$	tons per square kilometre per annum
t/a	tons per annum

1 INTRODUCTION

The Department of Water Affairs (DWA) appointed **BKS (Pty) Ltd** in association with three sub-consultants **Africa Geo-Environmental Services, MM&A and Urban-Econ** with effect from 1 December 2011 to undertake the **uMkhomazi Water Project Phase 1: Module 1: Technical Feasibility Study Raw Water** study.

On 1 November 2012, BKS (Pty) Ltd was acquired by **AECOM Technology Corporation**. As a result of the change in name and ownership of the company during the study period, all the final study reports will be published under the AECOM name.

In 2010, the Department of Arts and Culture published a list of name changes in the Government Gazette (GG No 33584, 1 October 2010). In this list, the Mkomazi River's name was changed to the **uMkhomazi River**. *The published spelling will thus be used throughout this technical feasibility study.*

1.1 BACKGROUND TO THE PROJECT

The current water resources of the Mgeni Water Supply System (WSS) are insufficient to meet the long-term water demands of the system. The Mgeni WSS is the main water source that supplies about six million people and industries in the eThekwini Municipality, uMgungundlovu District Municipality (DM) and Msunduzi Local Municipality (LM), all of which comprise the economic powerhouse of the KwaZulu-Natal Province.

The Mgeni WSS comprises the Midmar, Albert Falls, Nagle and Inanda Dams in KwaZulu-Natal, a water transfer scheme from the Mooi River and the newly constructed Spring Grove Dam. The current system (Midmar, Albert Falls, Nagle and Inanda dams and the MMTS-1) has a stochastic yield of 334 million m^3/annum (measured at Inanda Dam) at a 99% assurance of supply. The short-term augmentation measure, Phase 2 of the Mooi Mgeni Transfer Scheme (MMTS-2), the recently constructed Spring Grove Dam, will increase water supply from the Mgeni system by 60 million m^3/year. However, this will not be sufficient to meet the long-term requirements of the system.

Pre-feasibility investigations indicated that the development of the undeveloped uMkhomazi River, to transfer water to the existing Mgeni system, most likely will fulfil this requirement. The uMkhomazi River is the third-largest river in KwaZulu-Natal in terms of mean annual runoff (MAR).

Eight alternative schemes were initially identified as possible alternatives, and the Impendle and Smithfield scheme configurations have emerged as suitable for further investigation. The pre-feasibility investigation, concluded in 1998, recommended that the Smithfield Scheme be taken to a detailed feasibility-level investigation as its transfer conveyances would be independent of the existing Mgeni System, thus reducing the risk of limited or non-supply to eThekwini and some areas of Pietermaritzburg, and providing a back-up to the Mgeni System.

The ***Mkomazi-Mgeni Transfer Pre-feasibility Study*** concluded that the first phase of the uMWP would comprise a new dam at Smithfield on the uMkhomazi River near Richmond, a multi-level intake tower and pump station, a water transfer pipeline/tunnel to a balancing dam at Baynesfield Dam or a similar in-stream dam, a water treatment works at Baynesfield in the uMlaza River valley and a gravity pipeline to the Umgeni bulk distribution reservoir system, below the reservoir at Umlaas Road. From here, water will be distributed under gravity to eThekwini and possibly low-lying areas of Pietermaritzburg. Phase two of the uMWP may be implemented when needed, and could comprise the construction of a large dam at Impendle further upstream on the uMkhomazi River to release water to the downstream Smithfield Dam. Together, these developments have been identified as having a 99% assured stochastic yield of about 388 million m^3/year.

The DWA aims to have this scheme implemented by 2023.

1.2 OBJECTIVE, SCOPE AND ORGANISATION OF THE STUDY

According to the Terms of Reference (November 2010), the objective of the study project is to undertake a feasibility study to finalise the planning of the proposed uMkhomazi Water Project (uMWP) at a very detailed level for the scheme to be accurately compared with other possible alternatives and be ready for implementation (detailed design and construction) on completion of the study.

The feasibility study has been divided into the following modules, which will run concurrently:

- Module 1: Technical Feasibility Raw Water (DWA) (*defined below*);
- Module 2: Environmental Impact Assessment (DWA); and

- Module 3: Technical Feasibility Potable Water (Umgeni Water) (ranging from the WTW to the tie-in point with the eThekwini distribution system).

The layout as per module is shown in **Figure 1.1**.

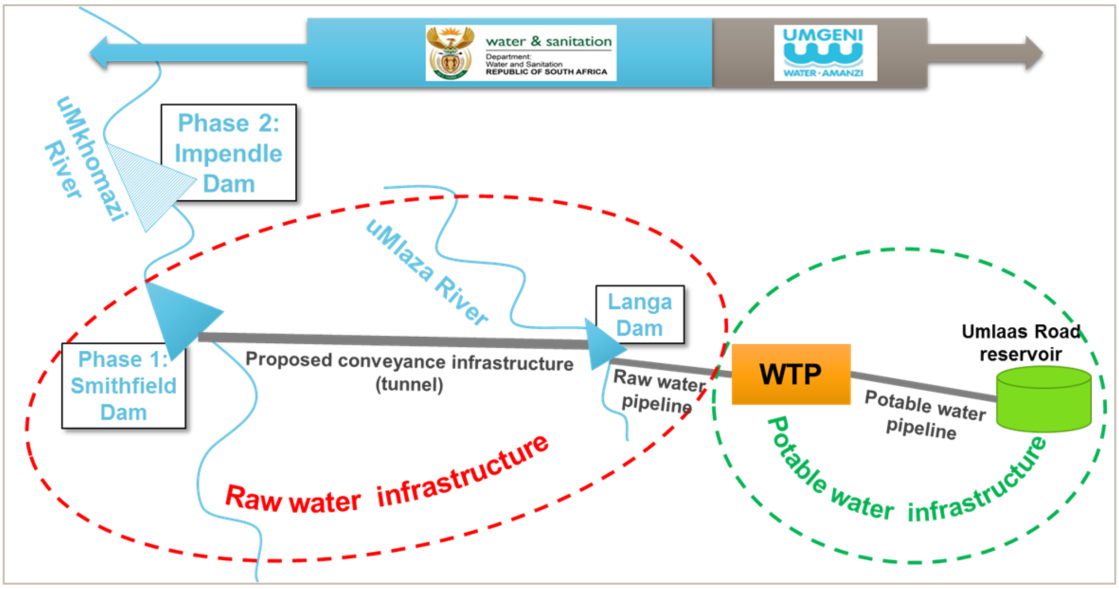

Figure 1.1: uMWP feasibility modules

This module, the raw water technical feasibility study, considers water resources aspects, engineering investigations and project planning and scheduling and implementation tasks, as well as an environmental screening and assessment of socio-economic impacts of the proposed project.

Some specific objectives for this study, recommended in the *Mkomazi-Mgeni Transfer Scheme Pre-feasibility* Study are listed below:

- Smithfield Dam (Phase 1) to be investigated to a detailed feasibility level;
- Investigate the availability of water from Impendle Dam (Phase 2) as a future resource to release to Smithfield Dam, and refine the phasing of the selected schemes;
- Optimise the conveyance system between Smithfield Dam and the proposed Baynesfield WTW;
- Undertake a water resources assessment of the uMkhomazi River Catchment, including water availability to the lower uMkhomazi;
- Evaluate the use of Baynesfield dam as a balancing dam; and
- Investigate the social and economic impact of the uMWP.

This study was undertaken in close collaboration with the DWA, UW and the Professional Services Providers (PSPs) of the other modules.

1.3 STUDY AREA

The study focus and key objective are related to the feasibility investigation of the Smithfield Dam and related raw water conveyance infrastructure. However, this is a multi-disciplinary project with the study area defined as the uMkhomazi River catchment, stretching to the north to include the uMngeni River catchment (refer to **Figure 1.2**). The various tasks have specific focus area, defined as:

- Water Resources: uMkhomazi and uMngeni River catchments;
- Water requirements: water users in the Mgeni WSS and the uMkhomazi River catchment;
- Engineering Investigations: proposed dams, Impendle (only for costing purposes), Smithfield and Langa Dams and the raw water conveyance infrastructure corridor between Smithfield Dam and the WTW of Umgeni Water at Bayensfield;
- Environmental screening as input for the Environmental Impact Assessment; and
- Socio-economic impact assessment: regional, provincial (KwaZulu-Natal (KZN)) and national.

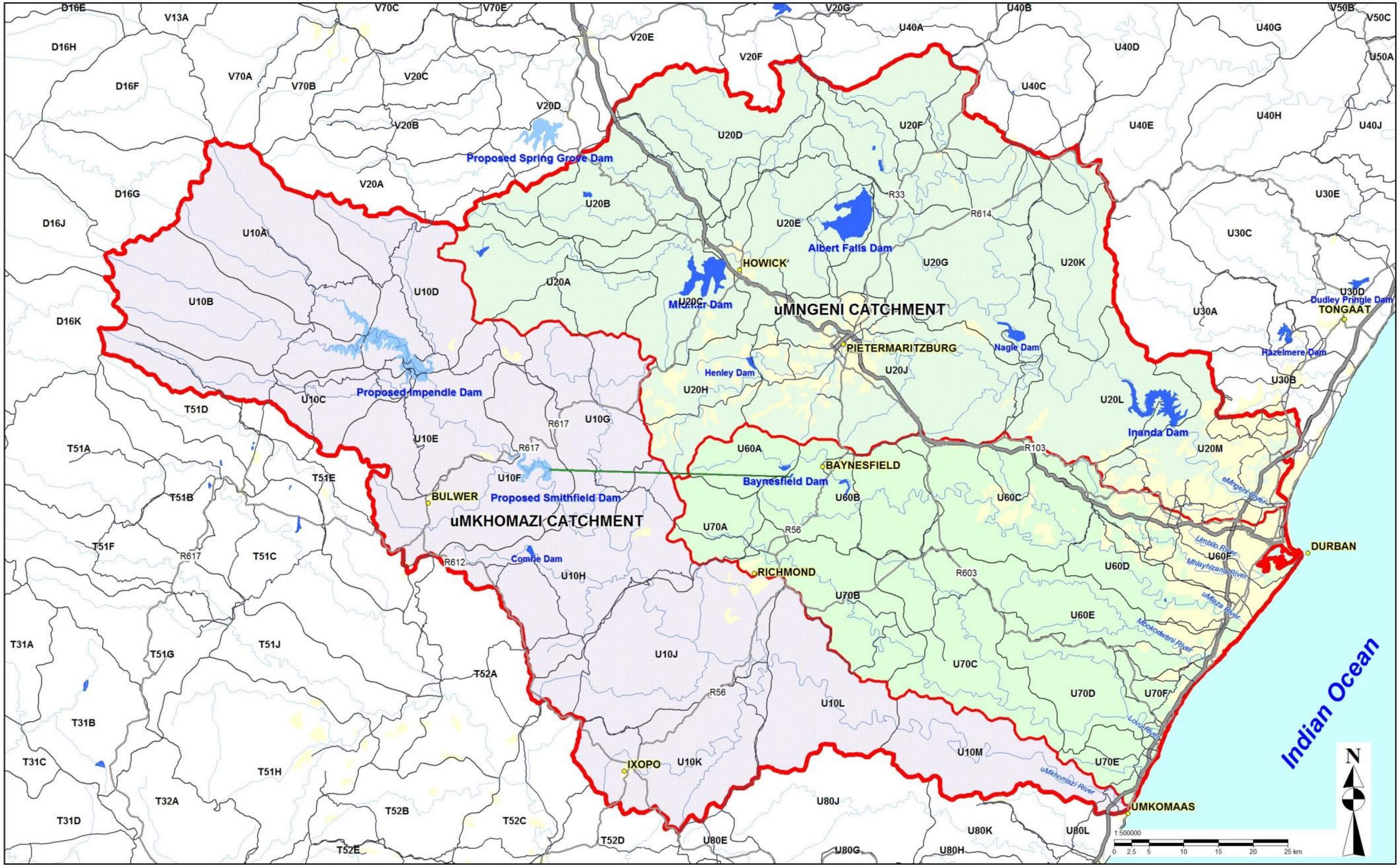

Figure 1.2: Locality map: study area of the uMWP

1.4 SCOPE OF THIS REPORT

This report covers the feasibility design of the selected scheme for the transfer of water from the uMkhomazi River to the Mgeni WSS to meet the UW water requirements below Umlaas Road, excluding the very north and south coast parts of the UW's supply area.

It includes a description of the selected scheme and provides the feasibility design aspects of all scheme components to provide a base for cost estimations. The engineering feasibility study was carried out in the following phases:

- Geotechnical
  - Seismic hazard potential
  - Seismic refraction investigation
  - Smithfield Dam, drilling and materials investigation
  - Langa Dam, drilling and materials investigation
  - Conveyance system, drilling and investigation
- Hydrology
  - Flood hydrology
  - Water resources hydrology
- Topographical survey
- Hydropower evaluation
- Feasibility design
  - Optimisation of conveyance
  - Dam position
  - Optimisation of scheme configuration
  - Dam type selection
  - Dam design (Smithfield and Langa Dams)
  - Conveyance design
- Economic Assessment
- Institutional aspects

1.5 STRUCTURE OF THIS REPORT

This report addresses the feasibility design phase of the project and has been structured as follows:

- Section 2: Description of the selected scheme
- Section 3: Design of Smithfield Dam
- Section 4: Design of the uMkhomazi uMlaza tunnel
- Section 5: Design of Langa Dam
- Section 6: Design of the pipe line connecting the tunnel outlet, Langa Dam and the WTW
- Section 7: Evaluation of hydropower potential
- Section 8: Design of gauging weirs
- Section 9: Design of the access roads
- Section 10: Layout and design of waste disposal sites
- Section 11: Description of Quarries and Borrow areas
- Section 12: Project power requirements
- Section 13: Project cost estimation
- Section 14: Estimation of construction programme
- Section 15: Cost estimation of phase 2 of Impendle Dam
- Section 16: Conclusions and recommendations
- Section 17: References

2 DESCRIPTION OF SELECTED SCHEME

2.1 LAYOUT

The pre-feasibility layout of the scheme has been improved and changed to include the findings of the geotechnical investigations as well as some optimization conducted as part of this feasibility study as described in the List of Reports. The scheme consists of the Smithfield Dam, uMkhomazi to uMlaza transfer tunnel, Langa Dam as well as the Baynesfield pipeline. The subcomponents of the scheme are shown in **Figure 2.1** and **Table 2.1**.

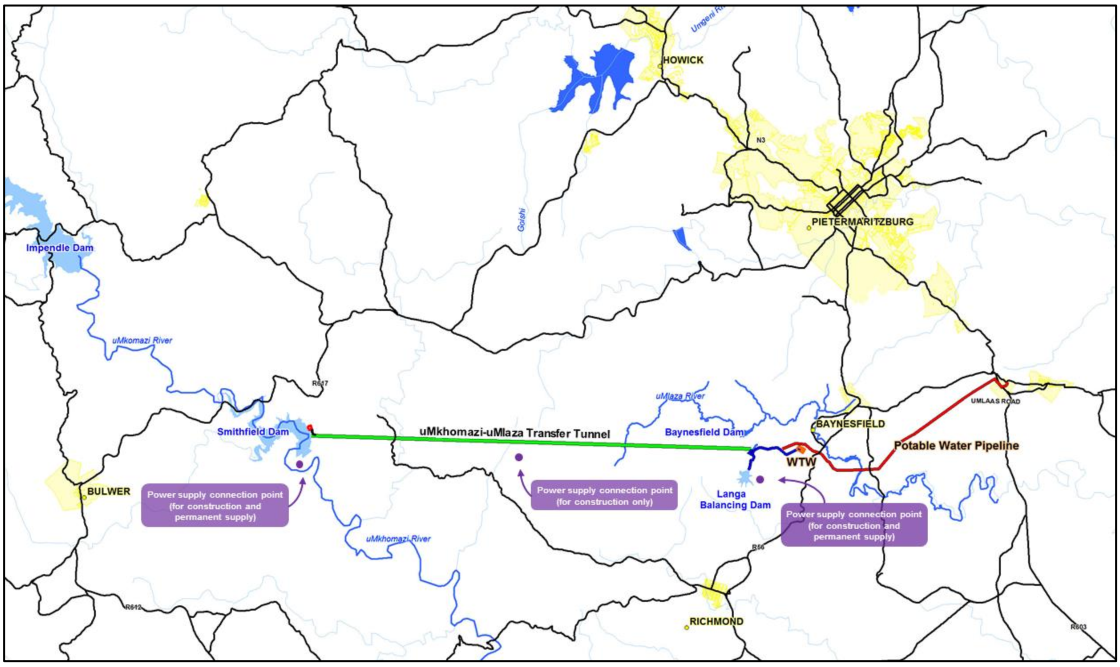

Figure 2.1: Selected Scheme Layout

Table 2.1: Components of the scheme

Major component	Subcomponent
Smithfield Dam	Smithfield Dam Flow Gauging Weir(s) Smithfield Tunnel Intake Tower Impendle Flow Gauging Weir Access Roads Deviation of Public Roads Deviation of Transmission Lines Smithfield Dam Hydropower Plant
uMkhomazi – uMlaza Tunnel	Smithfield Waste Disposal Site Tunnel Access Adits Access Roads Ventilation Shafts Central Waste Disposal Site
Baynesfield raw water pipeline	Baynesfield Hydropower Plant Access Roads
Langa Dam	Access Roads

2.2 BACKGROUND TO LAYOUT

The most important aspects identified which influence the size and layout of the scheme are summarised below:

- The UW's water requirements below Umlaas Road.
- The 1:200 year assured yield after meeting water reserve requirements.
- The lowest unit reference value size of Smithfield Dam. This was determined at 31% Mean Annual Runoff Storage volume.
- One 3.5 m diameter uMkhomazi – uMlaza pressure tunnel for the transfer of water as opposed to a considered pumping scheme. The tunnel exit is at Mbangweni Dam.
- A 2.6 m diameter raw water pipeline connecting the tunnel outlet with the treatment plant.
- The Langa Dam, located upstream of the Mbangweni Dam, which should provide supply for three weeks when inspection and maintenance of the tunnel is executed.

2.3 PRINCIPAL PROJECT DATA

The principal project data for each structure are summarised at the end of each section describing the design of the section.

3 SMITHFIELD DAM

3.1 SIZING BACKGROUND

Based on a detailed geotechnical investigation and using available materials at lowest cost, the *Dam Type Selection Report (Engineering Feasibility Design Report: Supporting Document 5 (P WMA 11/U10/00/3312/3/1/5))* recommended the following characteristics for Smithfield Dam:

- Full supply level (FSL) of 930 masl;
- Main earth core rockfill dam (ECRD) with a side channel spillway on the left bank; and
- Earthfill saddle embankment dam.

Three-dimensional illustrations of the proposed Smithfield Dam layout can be seen in **Figure 3.A.1** to **Figure 3.A.3** in **Annexure 3.A**. These illustrations portray what Smithfield Dam will look like after construction is finished, based on the feasibility design of the project.

3.2 FLOODS AND FLOOD HYDROGRAPHS

3.2.1 General

The main purpose of a hydrological and associated flood peak analysis is to determine a representative flood for various return periods.

Flow gauging weir U1H005 (refer to **Figure 3.1**) with latitude 29°44', longitude 29°54' and catchment area 1 744 km^2 is located approximately 11.4 km upstream of the proposed Smithfield Dam site.

Figure 3.1: Flow gauging weir U1H005

3.2.2 Maximum flood peak determination

Three methods can be used to determine the maximum flood peaks, namely the statistical, the TR137 (Kovacs, 1988) and the HRU (Midgley, 1972) methods. The TR137 determines the regional maximum flood (RMF) and the HRU method the probable maximum flood (PMF).

The RMF was determined with a K factor of 5 and the corresponding formula of:

$$Q = 100 \times Ae \times 0.5 \qquad \textbf{(Equation 3-1)}$$

Where:

Q = flow (m^3/s)

Ae = catchment area (km^2)

The RMF+Δ was determined with a K factor of 5.5, and the results of this method are shown in **Table 3.6**.

The PMF results are shown in **Table 3.1** and the determined PMF is 6 185 m^3/s,

The RMF+Δ value was used as the safety evaluation flood (SEF), as determined by the SANCOLD bulletin on the guideline relating to safety concerning floods (South African National Committee on Large Dams, 1990).

Table 3.1: Results of PMF calculations

Storm duration (h)	6	8	10	12	16	18	24
Effective probable maximum precipitation (mm)	124.7	136.6	157.8	179.9	196.4	206.1	230
Unit hydrograph Q (m³/s)	44.2	45.3	36.2	33.2	28.3	26.4	21.7
PMF (m³/s)	5 512	**6 185**	5 713	5 966	5 563	5 436	4 996

3.2.3 Historical floods

The existing DWA rating curve for flow gauge U1H005 has only been calibrated up to a flow depth of 2.71 m, correlating to a discharge of 637.8 m³/s. The annual series of maximum discharge values at U1H005 indicates that four flood events, summarised in **Table 3.2**, exceeded the maximum rating depth at the flow gauge.

Table 3.2: Exceedance of flow gauge rating depth during floods

Date of flood	Maximum depth of flow (m)
06-02-1976	2.823
29-09-1987	5.275
25-02-1988	3.695
27-01-1996	3.067

The 1976 and 1987 floods were previously determined by DWA as 1 000 m³/s and 2 770 m³/s, respectively *(DWA, 1987)*.

3.2.4 Hydrological analysis

Given the available historical flow record from 1961 to 2012 as recorded and compiled by DWA for flow gauging weir U1H005, a detail statistical analysis was conducted using the relevant data to determine flood peaks and compared with other approaches.

a) *Statistical analysis of recorded flow data*

In order to estimate the peak discharge of the floods listed in **Table 3.2**, the DWA rating curve was extrapolated using a “best-fit” polynomial trend line, shown in **Figure 3.2** as Option 1.

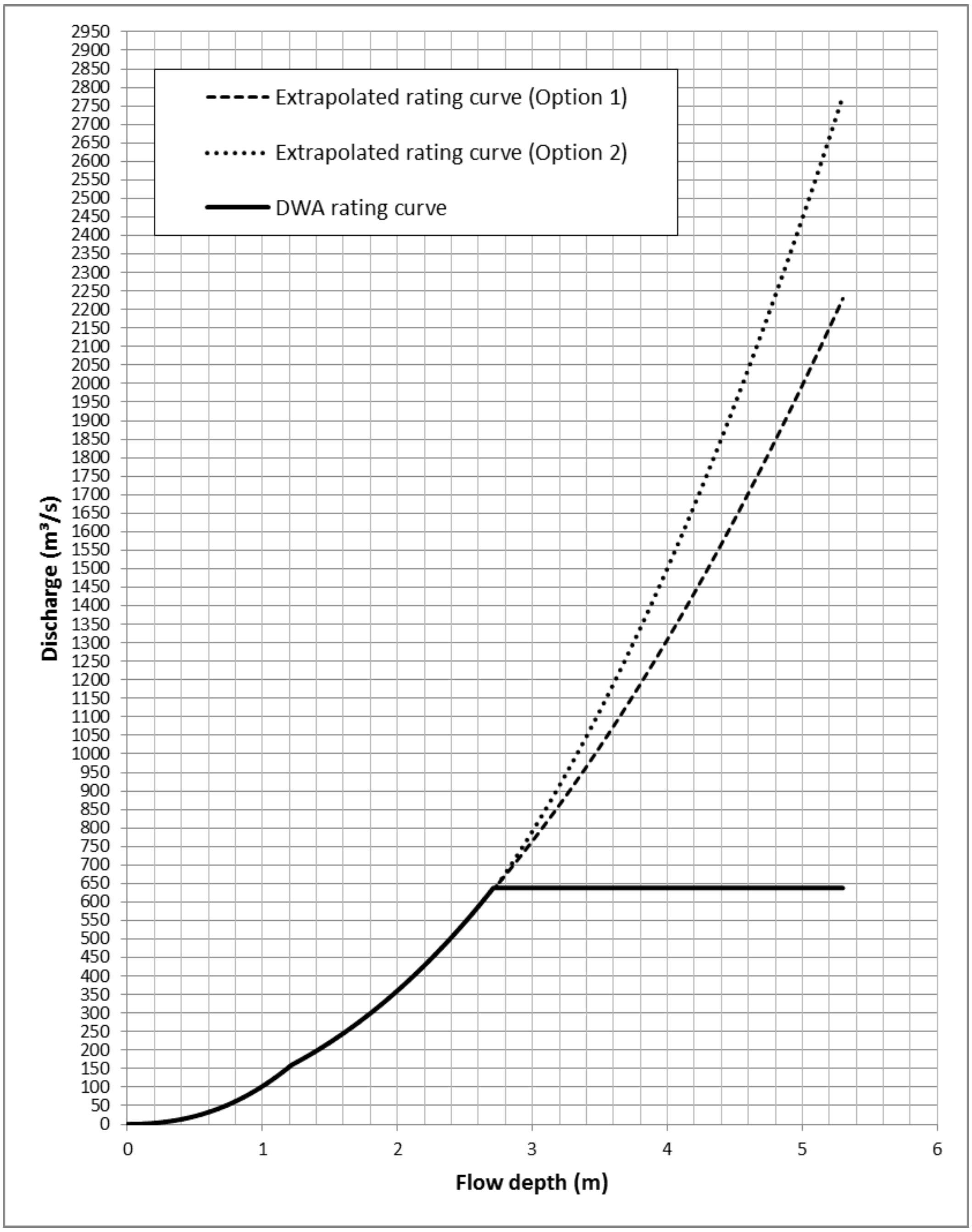

Figure 3.2: Flow gauging weir U1H005 rating curve

Technical Report 139 (DWA, 1987) estimated the 1987 peak discharge at U1H005 to be 2 770 m³/s, and based on this value the rating curve was extrapolated with an adjusted trend line (Option 2) to accommodate this value. The peak discharge values determined based on the trend lines (Options 1 and 2) are listed in **Table 3.3** as follows:

Table 3.3: Floods exceeding the flow gauge rating depth

Date of flood	Maximum depth of flow (m)	Peak discharge using extrapolated rating curve Option 1 (m^3/s)	Peak discharge using extrapolated rating curve Option 2 (m^3/s)
06-02-1976	2.823	682	688
29-09-1987	5.275	2 214	2 770
25-02-1988	3.695	1 131	1 264
27-01-1996	3.067	798	833

Following from **Figure 3.2** the annual maximum flood peak data was analysed in terms of extreme value (Gumbel and generalised for untransformed data) and log-normal (transformed data) distributions using the software *Utility Programs for Drainage (Sinotech, 2007)*. As a result, the associated flood peaks for various return intervals are summarised in **Table 3.4**.

Table 3.4: Results of statistical analysis of flood values at flow gauging weir U1H005

Return period (years)	Flood peak discharge (m^3/s)			
	Extreme value (Gumbel) (EV1)	Generalised exteme value (GEV)	Log-normal (LN)	Average
2	341	309	320	323
10	916	817	752	828
20	1 136	1 077	956	1 056
50	1 420	1 486	1 257	1 388
100	1 633	1 856	1 515	1 668
200	1 845	2 290	1 790	1 975
500	2 125	2 980	2 187	2 430
1 000	2 337	3 609	2 515	2 820
10 000	3 039	6 599	3 829	4 489

It follows from the results of **Table 3.4** that the flood peak discharge values differ for the various methods. The average flood peak values are proposed for use.

Based on these average values the respective flood peak flows for Smithfield Dam, as summarised in **Table 3.5**, were determined by extrapolating the relevant U1H005 flood peak values with the following formula:

$$Q_{Dam} = Q_{U1H005}\sqrt{\frac{A_{Dam}}{A_{U1H005}}} \qquad \textbf{(Equation 3-2)}$$

Where:

Q = flood peak discharge (m³/s)

A = catchment area (km²)

A_{Dam} = 2 058 km²

A_{U1H005} = 1 744 km²

Table 3.5: Representative flood peak values at dam sites based on a statistical analysis of recorded river flow data

Return period T (years)	Flood peak Q_T (m³/s)
	Smithfield Dam
2	351
10	900
20	1 147
50	1 507
100	1 812

b) *Comparative flood peak analysis*

A comparison of the flood peak values as determined by the statistical analysis of recorded river flow data at flow gauging weir U1H005 with those determined by deterministic and empirical approaches is provided in **Table 3.6**. The flood peak Q_T refers to a flood with a recurrence interval of one in T years. The following should be noted:

- The Rational Method was not used, as this method is recommended for catchments smaller than 15 km².
- The highest weighting (0.5) was applied to the Statistical Method since the results are based on observed floods at a flow gauging weir relatively close to the dam sites. The data spans a reasonable length of time (over

50 years) and is therefore considered very suitable for statistical analyses.

- A weighting of 0.2 was applied to the Alternative Rational Method, with no limitation on the catchment size when used. This method explicitly takes into account some of the most significant catchment specific factors influencing runoff, namely mean annual precipitation (MAP), topography, permeability and vegetation.
- The Unit Hydrograph, Empirical and Standard Design Flood methods were given a lower weighting compared to the Alternative Rational Method as they are based on regional rainfall-runoff relationships, which might not be equally applicable to all catchments within the defined regions.

Table 3.6: Summary of flood peak determination

Dam site	Catchment area (km²)	Recurrence period T (years)	Flood Peak Q_T (m³/s)						Final flood peak (m³/s)
			Statistical Method	Alternative Rational Method	Empirical Method (M&P)	Unit Hydrograph Method	Standard Design Flood Method	Regional Maximum Flood Method	
Smithfield	2 058	*Weighting*	*0.5*	*0.2*	*0.1*	*0.1*	*0.1*	*-*	
		2	351	441	-	300	84	-	336
		10	900	1 115	1 027	714	902	-	937
		20	1 147	1 474	1 393	999	1 363	-	1 244
		50	1 507	2 015	1 932	1 519	2 061	2 108	1 708
		100	1 812	2 524	2 437	2 122	2 654	2 567	2 389
		RMF	-	-	-	-	-	4 520*	4 537
		$RMF_{+\Delta}$							5 647
		PMF							6 185

**DWA Determination*

c) *Comparison with previous flood peak analyses*

A previous flood peak determination undertaken by the DWA in 1998 for the purposes of the *Mkomazi/Mooi-Mgeni Transfer Scheme Pre-Feasibility Study* was also based on a statistical analysis of available flow data for flow gauging weir U1H005. A comparison of the flood peaks with the previously determined DWA values is provided in **Table 3.7**.

Table 3.7: Comparative summary of flood peak determination based on annual flows

Dam site	Recurrence period T (years)	Flood peak Q_T (m³/s)		
		1998 Statistical Method (DWA)	2012 Statistical Method (Average)	2012 recommended flood peak
Smithfield	2	390	351	336
	10	1 000	900	937
	20	1 310	1 147	1 244
	50	1 750	1 507	1 708
	100	-	1 812	2 389
	200	2540	2 155	2 620
	RMF	4 520	-	4 540
	SEF ($RMF_{+\Delta}$)	-	-	5 647

Based on the comparison provided in **Table 3.7**, it follows that the 2012 flood peak values as presented in this report are, in general, a little higher than the 1998 DWA values. This may be due to the assumptions regarding the plot of a representative projection function.

3.2.5 Hydrographs

a) *Methodology*

The Synthetic Unit Hydrograph (SUH) method from the *HRU report 1/72* (Midgley, 1972) was used to determine the inflow hydrograph for the 200-year flood peak. The input data used for the hydrograph is shown in **Table 3.8** and the description of the determination of this data is shown in **Figure 3.B.1** and **Figure 3.B.2** in **Annexure 3 B**.

Table 3.8: Hydrograph input data

Area (km^2)	Slope (km/km)	Rainfall region	Veld type	Lc (km)	Tc (hour)
2 058	0.0061	Summer	5	71.345	18.9

The shape of the inflow hydrograph for the 200-year flood peak was used to determine the hydrographs for different flood peaks (ordinates were scaled pro-rata).

The following hydrographs were plotted and are shown in **Figure 3.3**:

- 2014 determined 1:200 year flood peak;
- DWA 1:200 year flood peak;
- 2014 determined RMF;
- 2014 determined RMF +Δ (SEF = 5 650 m^3/s);
- RMF Triangular flood peak;
- RMF +Δ (SEF) Triangular flood peak; and
- 2014 determined PMF.

The latest 200-year hydrograph determined compares well with the one determined by DWA in 1998. The AECOM SEF ($RMF_{+\Delta}$) hydrograph was used in the further calculations as the data of the PMF method had too many uncertainties.

The tables for the various hydrographs used are represented in **Table 3.B.1** and **Table 3.B.2** in **Annexure 3 B**.

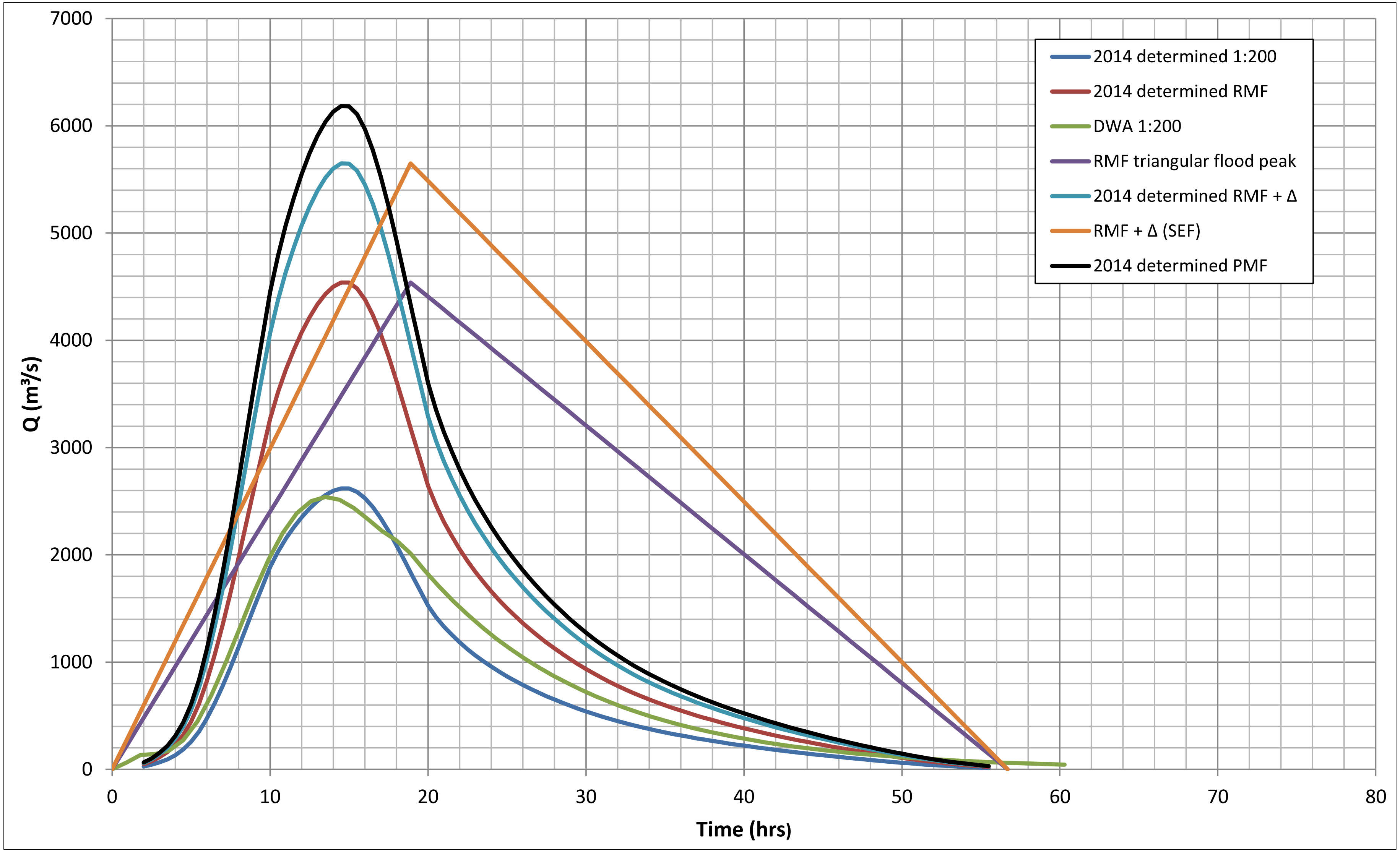

Figure 3.3: Smithfield Dam hydrographs

3.2.6 Winter (April to September) flood peak analysis

Streamflow records for flow gauging weir U1H005 were analysed regarding peaks for the months of April to September. The peak flows for these months were selected and analysed with the *Utility Program for Drainage* (Sinotech, 2007). Different statistical distributions were used to approximate the return periods of the available data. The fit of these distributions were verified by hand calculations and are:

- Log-normal;
- Log-extreme value type 1;
- Log-Pearson type 3; and
- Extreme value type 3.

These distributions were plotted onto the available data and visually inspected to obtain the distribution which fitted the data most closely. The results are shown in **Annexure 3 B** as **Figure 3.B.3** to **Figure 3.B.6**. The log-Pearson type 3 distribution fitted the data most closely and this distribution was used to determine the floods for these months as shown in **Table 3.9**.

Table 3.9: April to September flood peak results

Recurrence period T (years)	Flood Peak (m³/s)
2	38
5	74
10	107
20	145
50	206
100	261

No hydrographs were developed, as the benefit for flood attenuation was analysed and found to be insignificant.

3.3 STAGE-STORAGE VOLUME AND SURFACE AREA

The stage-storage volume and surface area relationship from the available contour map is shown in **Table 3.10** and **Figure 3.4**.

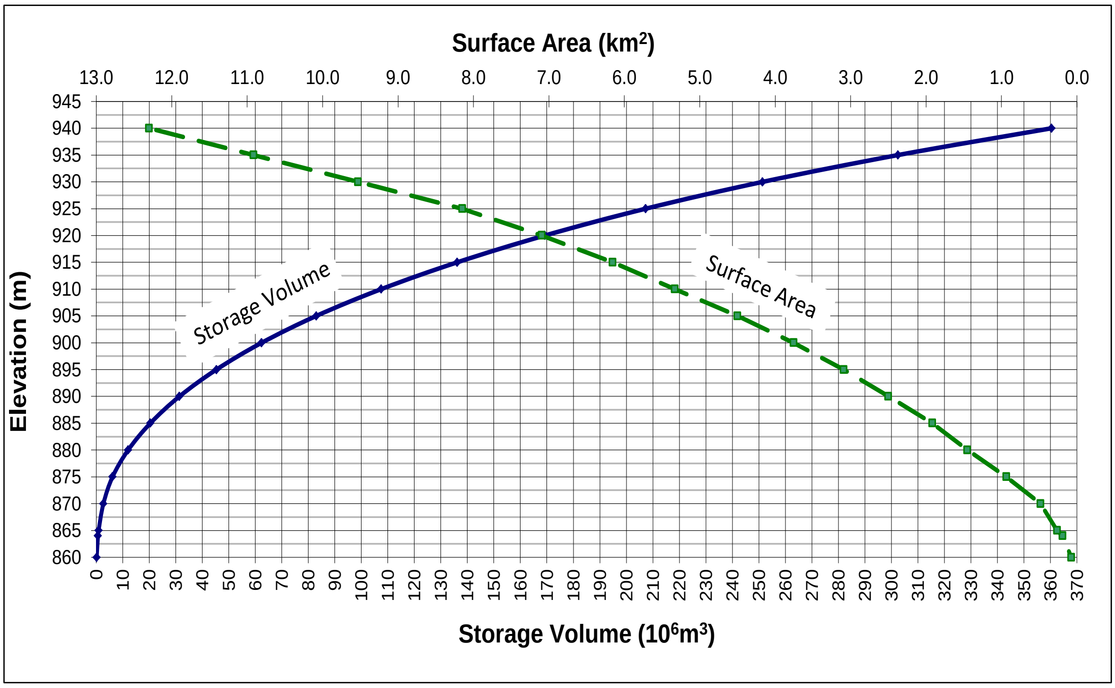

Figure 3.4: **Storage volume and surface area curves for the proposed Smithfield Dam**

Table 3.10: **Storage volume and surface area for the proposed Smithfield Dam**

Contour (masl)	Surface Area (km²)	Storage Volume (10^6 m³)
856	0.08	0.00
857	0.01	0.04
858	0.02	0.06
859	0.03	0.08
860	0.08	0.13
864	0.19	0.63
865	0.26	0.86
870	0.48	2.68
875	0.93	6.12
880	1.45	12.02
885	1.92	20.40
890	2.50	31.38
895	3.09	45.32
900	3.75	62.39
905	4.50	82.98
910	5.33	107.51
915	6.15	136.18
920	7.09	169.26
925	8.15	207.31

Contour (masl)	Surface Area (km²)	Storage Volume (10^6 m³)
930	9.53	251.43
935	10.91	302.48
940	12.30	360.46

3.4 FOUNDATION MATERIALS

3.4.1 General

The site comprises shales (mudrocks) with subordinate sandstones and intrusions of dolerite. Three near-horizontal dolerite sills have intruded mainly concordantly into the sedimentary strata and are responsible for the narrow river valley at the dam site and the presence of good quality rock for concrete aggregate and rockfill. The site has a low seismic risk.

3.4.2 Foundation

a) Main embankment

The founding level for the shells of the rockfill embankment is summarised as follows:

- At the upper left and right flanks a 6 to 10 m layer of colluvium and residual soil/completely weathered shale has to be removed;
- In the central river section 1.5 to 5 m of residual soil/completely weathered shale/dolerite and medium dense river alluvium has to be removed; and
- A large part of the right flank has 11.2 to 14.4 m of transported sandy clay with boulders which has to be removed.

The excavation for the founding level will yield a large volume of material, which might be suitable as impervious and semi-pervious earthfill for the saddle embankment. Laboratory testing of this material will have to be conducted to confirm the suitability.

The clay core of an earthfill or rockfill dam is normally founded on material that is either sufficiently impervious or can be rendered impervious by means of grouting.

It will be necessary to make provision for a grout curtain to a depth of about 66% of the water head along the centre line. Although grout penetration might be small except in local zones, the drilling, water test and grout records from a grouting operation are very important and can be considered the final stage of a geotechnical investigation when sub-surface information is obtained at close intervals below the footprint of the dam.

Excavation depths at borehole positions were based on the results of the geotechnical investigation carried out along the dam centre line

Table 3.11 summarises the proposed excavation depths. The locations of the various test pits and boreholes are shown in **Annexure 3 C** as **Figure 3.C.1** and **Figure 3.C.2**.

Table 3.11: Excavation depths for main earth core rockfill dam

Borehole No.	Elevation (masl)	Excavation depth (m)	
		Core	Shell
DLS 3	922.17	6.0	6.0
DL 1	918.23	10.6	10.3
DLS 2	914.34	8.4	8.4
DLS 1	904.25	4.0	3.0
DL 3	889.54	3.5	2.2
DL 4	879.25	2.0	1.5
DR 2	857.46	3.6	3.6
DR 1	857.32	10.0	5.0
DRS 1	885.58	4.5	4.5
DTS 1	888.42	5.2	5.2
DR 3	900.15	11.2	11.2
BH1004	901.20	12.5	12.5
DRS 2	903.81	15.0	14.4
DR 4	909.44	7.5	7.5
DRS 3	925.13	3.5	3.2

b) Saddle embankment

The founding level for the shells of an earth embankment is typically founded on material with low organic content, low compressibility and with shear strength similar to the dam wall material. This means that a 0.1 to 0.5 m thick layer of organic topsoil has to be removed along the centre line and that founding will take place on highly weathered shale.

The clay core of an earthfill or rockfill dam is normally founded on material that is either sufficiently impervious or can be rendered impervious by means of grouting. The clay core of an earthfill dam across the saddle embankment can be founded on moderately weathered shale that occurs at depths of between 2 and 4 m. This excavation depth will also be adequate for the concrete structure of the fuse plug.

If Quarry I is developed just upstream of the saddle embankment, the flow path underneath the embankment will be considerably shortened and it is recommended that provision be made for a grout curtain to a level at least 20 m below the quarry floor (approximately 845 masl).

Excavation depths at borehole positions were based on the results of the geotechnical investigation carried out along the dam centre line. **Table 3.11** summarises the proposed excavation depths. The locations of the various test pits and boreholes are shown in **Annexure 3 C** as **Figure 3.C.1** and **Figure 3.C.2**.

Table 3.12: Excavation depths for earth saddle embankment dam

Borehole no.	Elevation (masl)	Excavation depth (m)	
		Shell	Core
SSS1	930.2	0.5	2.0
SES1	917.4	1.5	3.2
SES2	911.9	0.5	3.0
SES3	915.2	0.5	2.5

c) *Spillway*

The position of the main spillway structure was not drilled for foundation levels and needs to be investigated during the tender and detail design phase.

The control structure for a side spillway on the upper left flank can be founded on slightly weathered shale at depths ranging between 15 and 20 m below ground surface and the concrete lined channel can be founded on moderately weathered shale at depths of between 10 and 12 m.

This excavation depth for the clay core or the saddle embankment will be adequate for the concrete structure of the fuse plug spillway.

3.5 RIVER DIVERSION

3.5.1 Introduction

The purpose of river diversion is to enable construction of the main dam embankment, especially in the river section, while accommodating the river flows and possible floods at an acceptable risk of delays and damages.

River diversion phases and associated structures are described in this section and are engineered along the following:

- Identification of the river diversion phases;
- Sizing of the diversion structures;
- Addressing of risk; and
- Construction programme.

The arrangement of this section is as follows:

- **Section 3.5.2**: River diversion philosophy in terms of the river diversion phases, structures and risk are summarised in **Table 3.13**, as well as the layout shown in **Figure 3.11**.
- **Section 3.5.3**: Description of diversion tunnels
- **Section 3.5.4**: Backwater analyses for determining the cofferdam crest levels.
- **Section 3.5.5**: Cofferdam characteristics.

Although one intake tower for both the river releases and the tunnel was considered in the "*Optimization of scheme configuration*" report, the most feasibility option is two intake works, one for the river releases at the Smithfield Dam wall and one for the for the uMkhomazi – uMlaza Tunnel.

3.5.2 River diversion philosophy

The river diversion period is planned over three years with closure for impoundment in the fourth year. The risk of flooding during the winter months (low flow season) is appreciably lower than during the summer months (high flow season). Therefore, to a great extent, the construction programme dictates the sizing of the different river diversion stages.

To ensure that the risk is within acceptable limits, the hydraulic sizing of the various river diversion stages must be seen as the minimum requirements.

The phases, seasons, description of the construction of associated structures and the risk of damages are given in **Table 3.13** to **Table 3.19**, and graphically illustrated in **Figure 3.5** to **Figure 3.11**.

Table 3.13: River diversion phase 1

River diversion phase	Season	Description of construction of structures	Risks associated with cofferdams and figures illustrating phase
Phase 1: Water in river	Summer of Year 1	Construct Cofferdam 1 and Cofferdam 2 and excavate the tunnel inlet and outlet portals. Excavate and provide rock support to the two 8 m diameter tunnels and line Tunnel 1. Construct the foundation of intake tower to dam bottom outlet. This includes for an access connection tunnel between the tunnels and a plug on the one side. Commence with provision of grout curtain of the main dam.	**Earthfill Cofferdam 1** and **Earthfill Cofferdam 2** are to accommodate the **1:10 year flood event** without overtopping. See **Figure 3.5**

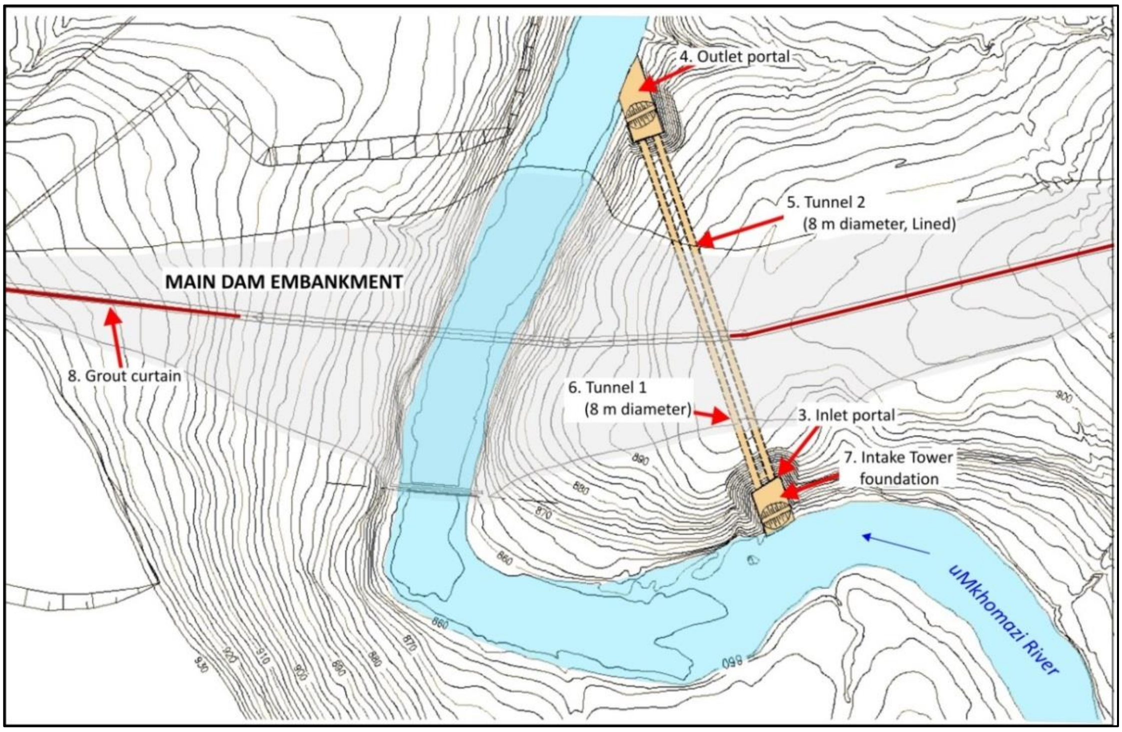

Figure 3.5: River diversion phase 1 – summer of year 1

Table 3.14: River diversion phase 2

River diversion phase	Season	Description of construction of structures	Risks associated with cofferdams and figures illustrating phase
Phase 2: Water diverted through tunnels	Winter of Year 1	Remove Cofferdams 1 and 2 and construct earthfill Cofferdam 3 in the river downstream of Cofferdam 1 to allow for the construction of Cofferdam 5. Construct Cofferdam 4 to prevent water exiting the tunnels to enter the main dam embankment area upstream in the river. Construct upstream concrete gravity Cofferdam 5. Proceed with lower parts of intake structure as well as parts of the main dam outside the river section.	**Earthfill Cofferdam 3** is to accommodate the **1:10 year winter flood event** without overtopping. **Cofferdam 4** to accommodate the **1:50 year flood event** without overtopping. **Concrete gravity Cofferdam 5** to accommodate the **1:20 year winter flood event** without overtopping. See **Figure 3.6**

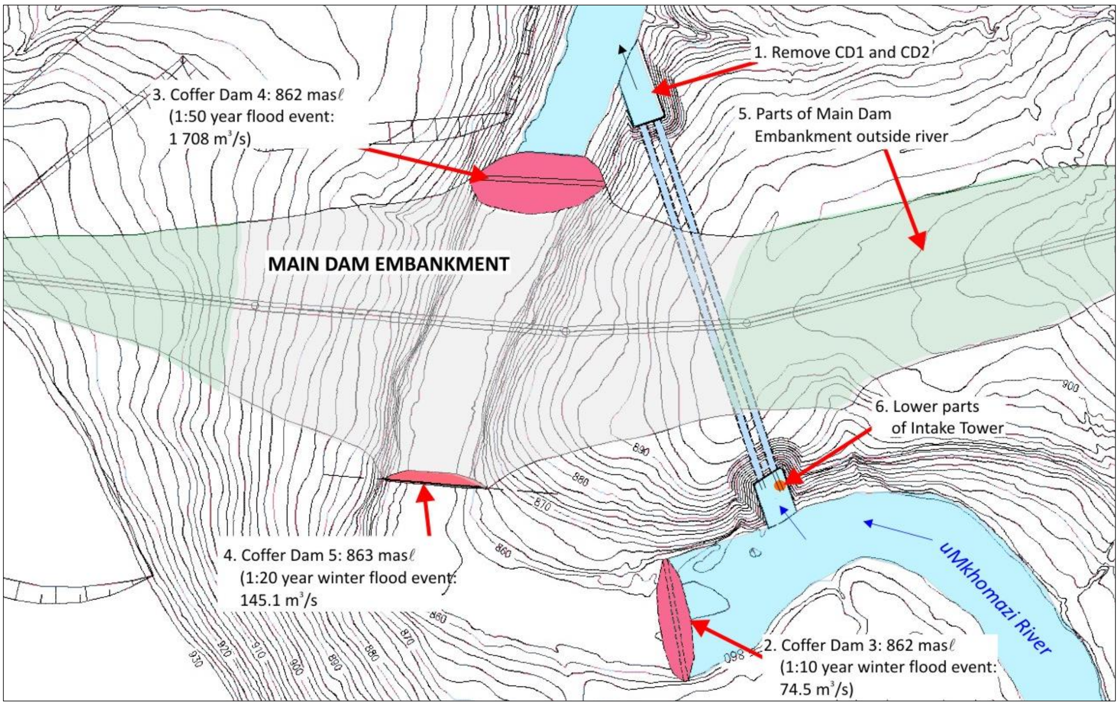

Figure 3.6: River diversion phase 2 – winter of year 1

Table 3.15: River diversion phase 3

River diversion phase	Season	Description of construction of structures	Risks associated with cofferdams and figures illustrating phase
Phase 3: Water diverted through tunnels and over Cofferdam 4	Summer of Year 2	Proceed with construction of Intake Tower. Provision of grout curtain in river section may commence.	See **Figure 3.7**

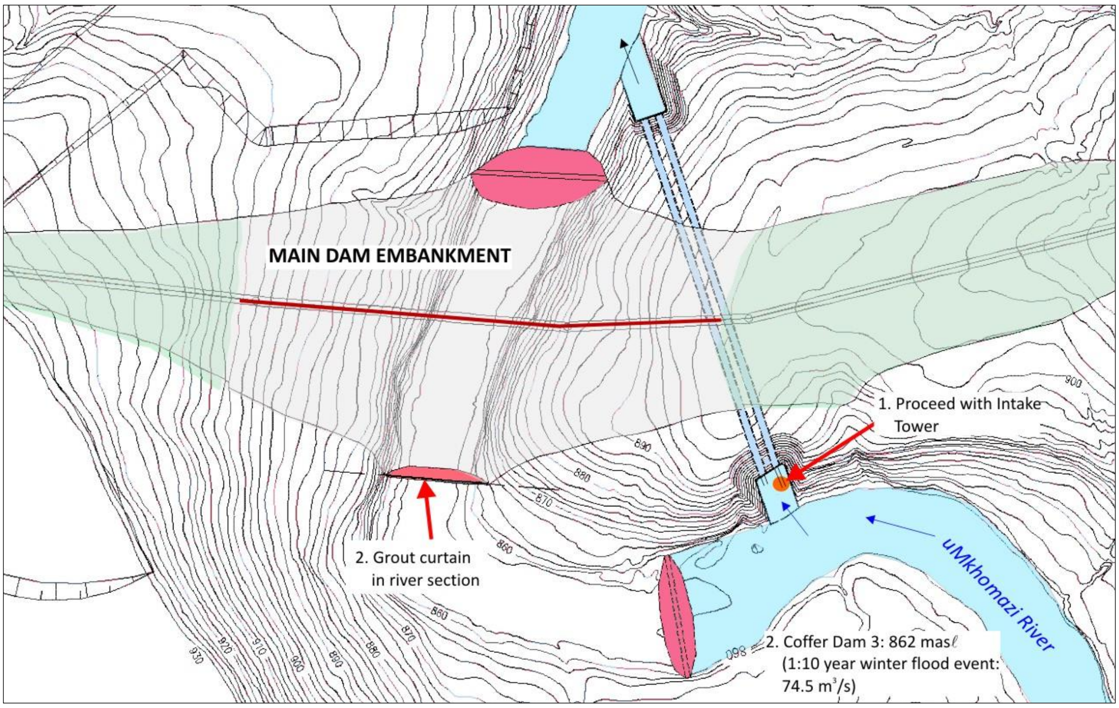

Figure 3.7: River diversion phase 3 – summer of year 2

Table 3.16: River diversion phase 4

River diversion phase	Season	Description of construction of structures	Risks associated with cofferdams and figures illustrating phase
Phase 4: Water diverted through tunnels	Winter of Year 2	Construct rockfill Cofferdam 6. Cofferdam 5 is abutting the rockfill of Cofferdam 6. Construct the upstream part of the main dam embankment in the uMkhomazi River on the downstream side of the gravity wall (Cofferdam 6). Proceed with the construction of the intake tower.	**Rockfill Cofferdam 6** to accommodate the **1:50 year summer flood event** without overtopping. See **Figure 3.8**

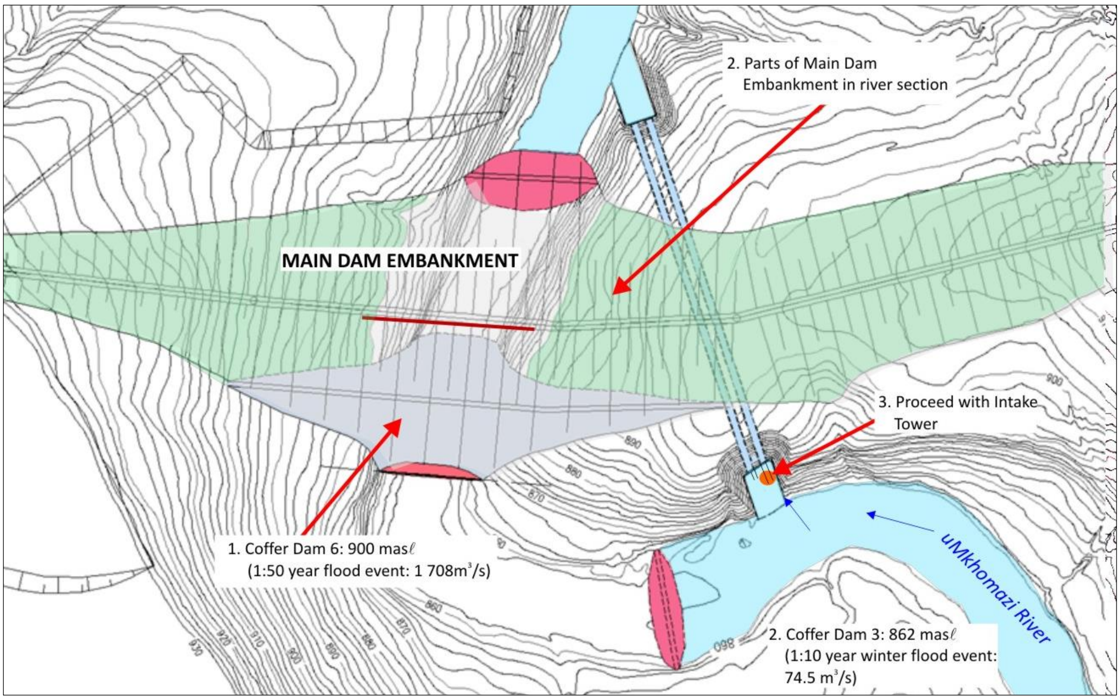

Figure 3.8: River diversion phase 4 – winter of year 2

Table 3.17: River diversion phase 5

River diversion phase	Season	Description of construction of structures	Risks associated with cofferdams and figures illustrating phase
Phase 5: Water diverted through tunnels	Summer of Year 3	Remove Cofferdam 3 and continue with the construction of main dam embankment in the uMkhomazi River section on the downstream side of the gravity wall. Complete intake tower to NOC level and complete access bridge.	See **Figure 3.9**

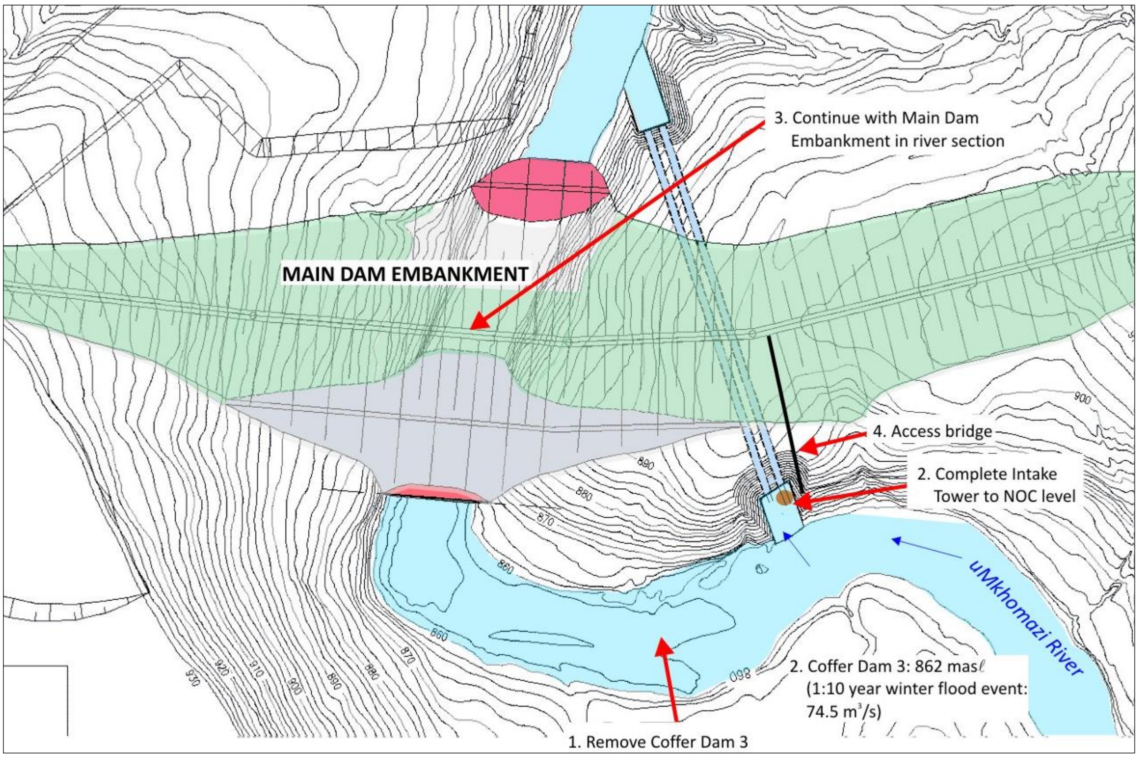

Figure 3.9: River diversion phase 5 – summer of year 3

Table 3.18: River diversion phase 6

River diversion phase	Season	Description of construction of structures	Risks associated with cofferdams and figures illustrating phase
Phase 6: Divert water through one tunnel	Winter of Year 3	Complete the remainder of the main dam embankment in the river section. Plug Tunnel 1 and insert the bottom part of the outlet pipes of the intake tower as well as the sleeve valves. Insert all butterfly valves. Complete control house on top of the intake tower.	See **Figure 3.10**

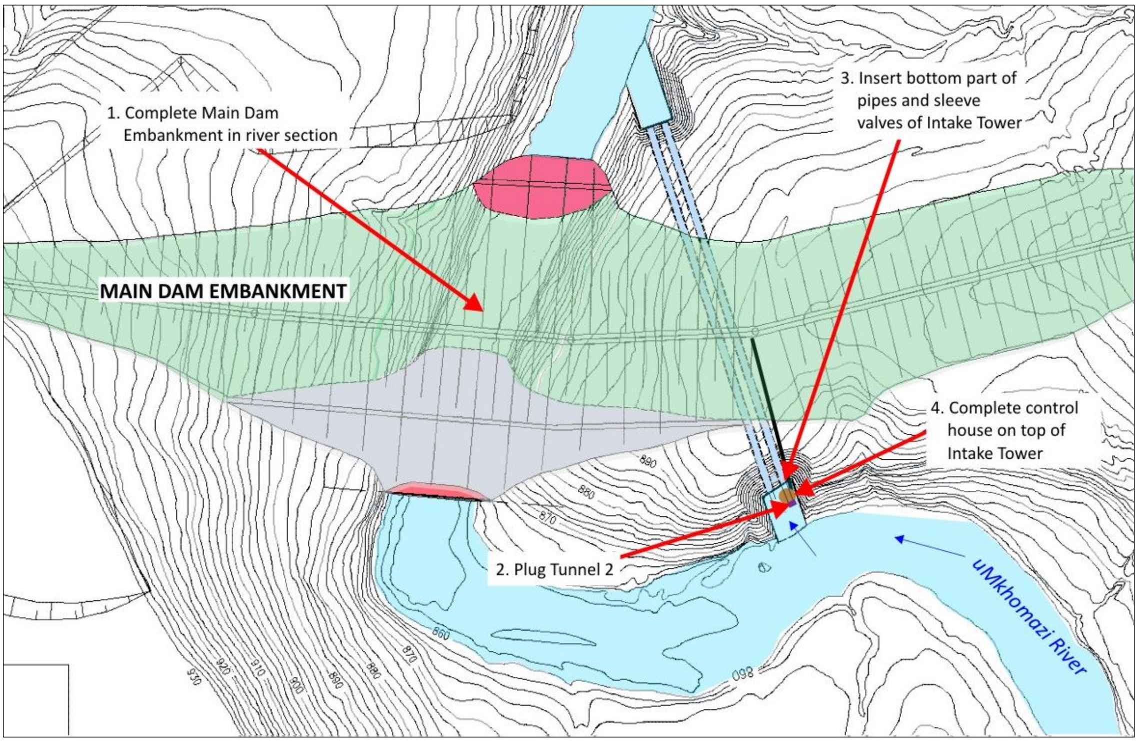

Figure 3.10: River diversion phase 6 – winter of year 3

Table 3.19: River diversion phase 7

River diversion phase	Season	Description of construction of structures	Risks associated with cofferdams and figures illustrating phase
Phase 7: Impoundment commencement	Summer of Year 4	Plug Tunnel 2. Reinstate access connection tunnel between Tunnel 1 and 2.	See **Figure 3.11**

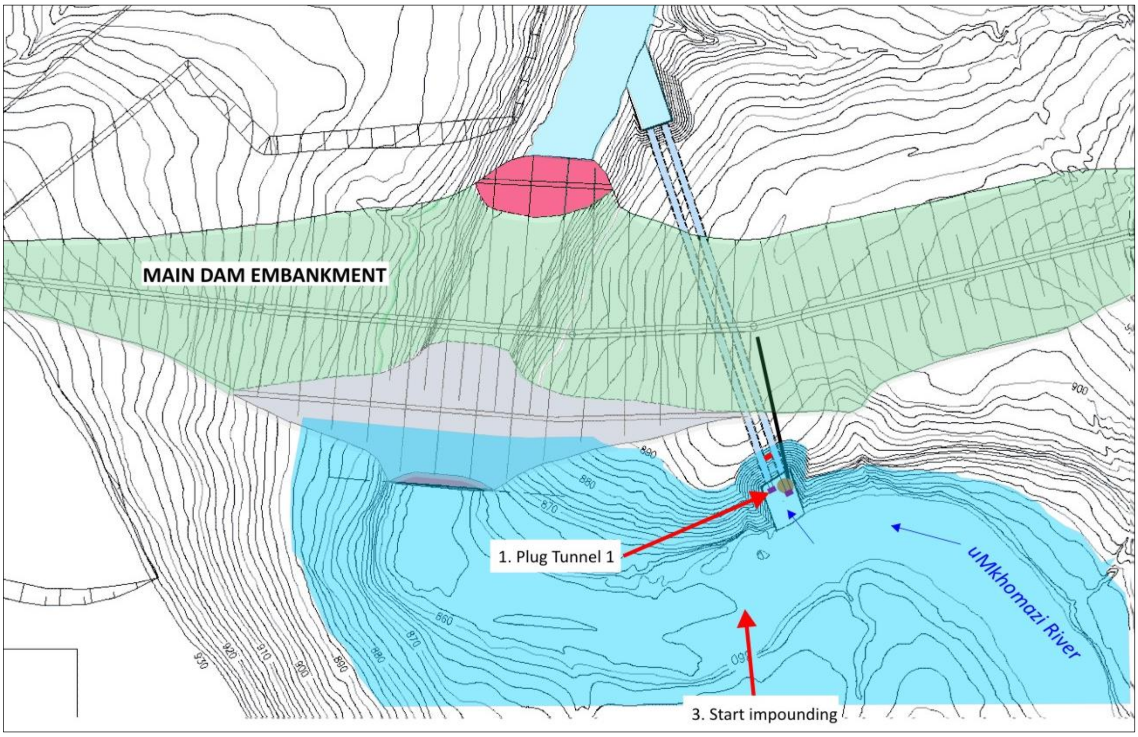

Figure 3.11: River diversion phase 7 – summer of year 4

3.5.3 Diversion tunnels

The purpose of the two diversion tunnels is to divert river flows and possible floods away from the construction area of the main dam embankment. After construction is completed, Tunnel 2 will serve as a permanent outlet to the uMkhomazi River, accommodating the outlet pipes from the outlet works on top of the tunnel.

Figure 3.12 shows the general layout of the Tunnel 1 and Tunnel 2 bellmouth intake sections. The upstream view of the tunnel intakes is shown in **Figure 3.13**.

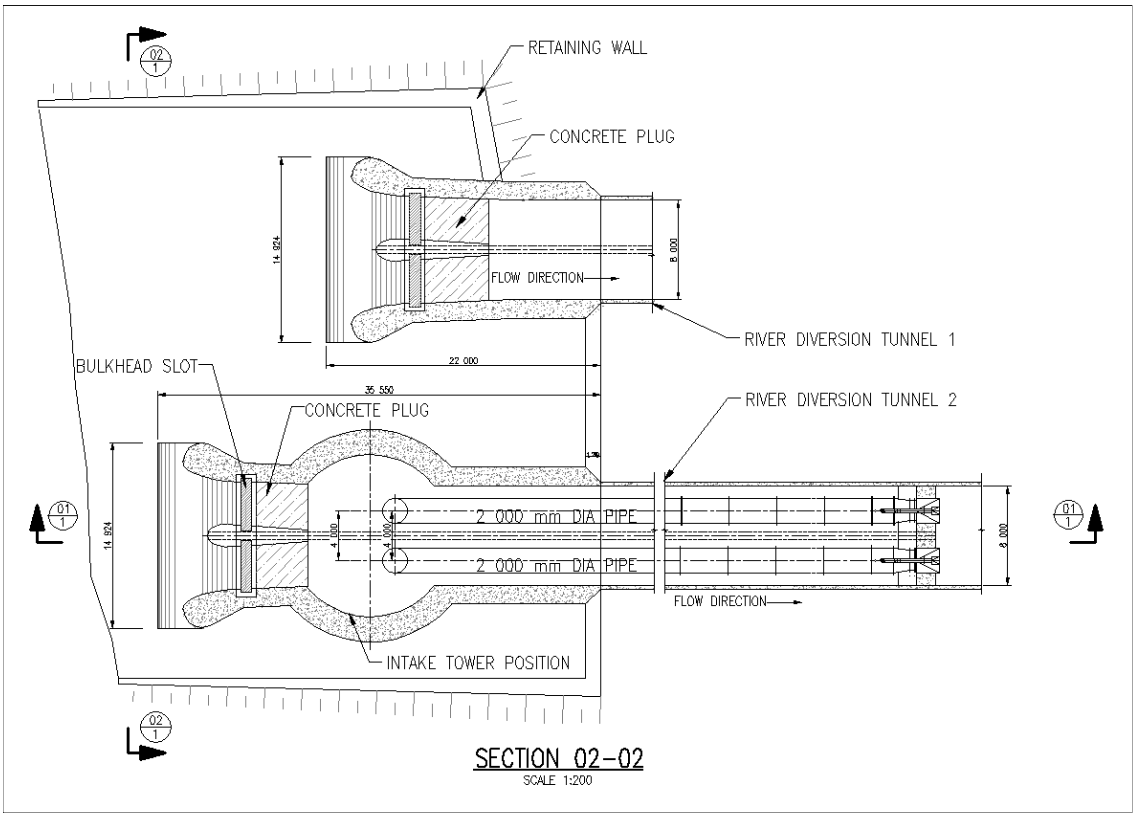

Figure 3.12: Plan layout of intake tunnels

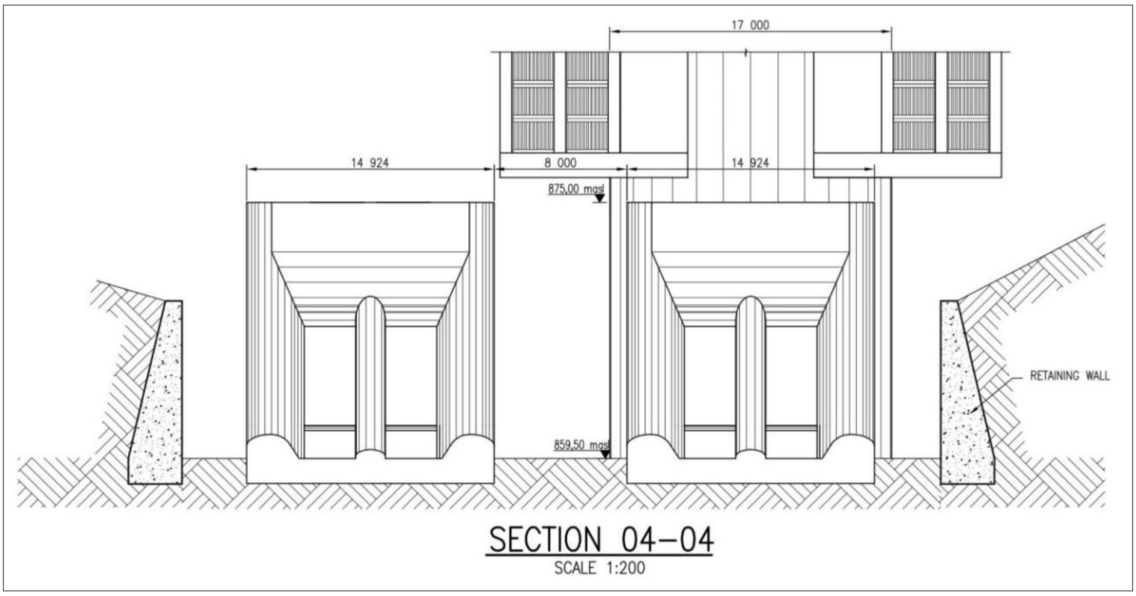

Figure 3.13: Upstream view of tunnel outlet

a) Tunnel 1

The cross-section and internal dimensions of Tunnel 1 is shown in **Figure 3.14** and a slope of 1V:100H to convey water during river diversion. The length of the tunnel is about 390 m. The invert level of the tunnel inlet is at 859.5 masl and the invert level for the outlet at 855.5 masl. The inlet of the tunnel is bell mouthed with concrete to smooth flow lines and hence to reduce hydraulic losses at the entrance.

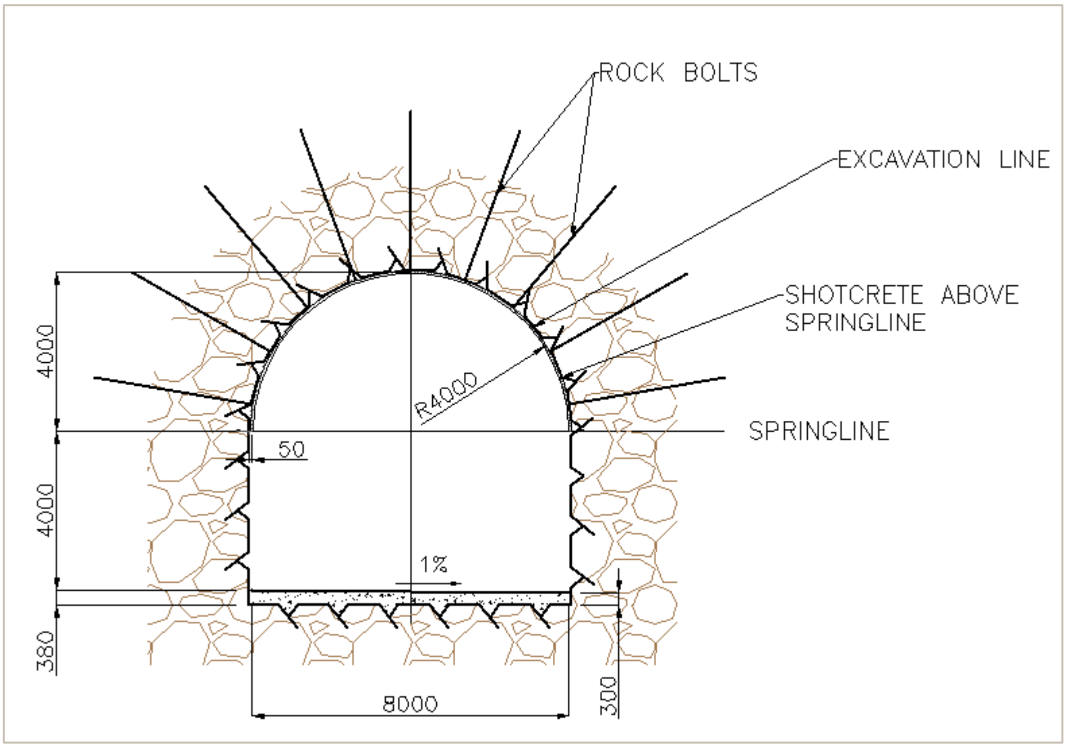

Figure 3.14: Tunnel 1 cross-section

Tunnel 1 has the same geometrical properties as Tunnel 2. However, the intake section of Tunnel 1 will be shorter and will not be lined over its entire length. Tunnel 1 will be plugged during Phase 7 of the river diversion to allow impoundment to commence.

b) Tunnel 2

Tunnel 2 is a concrete lined tunnel with an internal diameter of 8 m and a slope of 1V:100H to convey water during river diversion. The length of the tunnel is about 390 m. The invert level of the tunnel inlet is at 859.5 masl and the invert level for the outlet at 855.5 masl. The inlet of the tunnel is bell mouthed with concrete to smooth flow lines and hence to reduce hydraulic losses at the entrance.

Tunnel 2 will serve as a permanent outlet accommodating the outlet pipes leading from the outlet works on top of the tunnel to a position in the tunnel. The access to the valves will be from the intake tower.

The tunnel will be plugged during Phase 6 of the diversion works to allow for the installation of the bottom part of the outlet pipes of the intake tower and the sleeve valves. Bulkhead slots downstream of the bellmouth intake allow for the tunnel to be closed off prior to plugging. The cross-section of Tunnel 2 is illustrated in **Figure 3.15**.

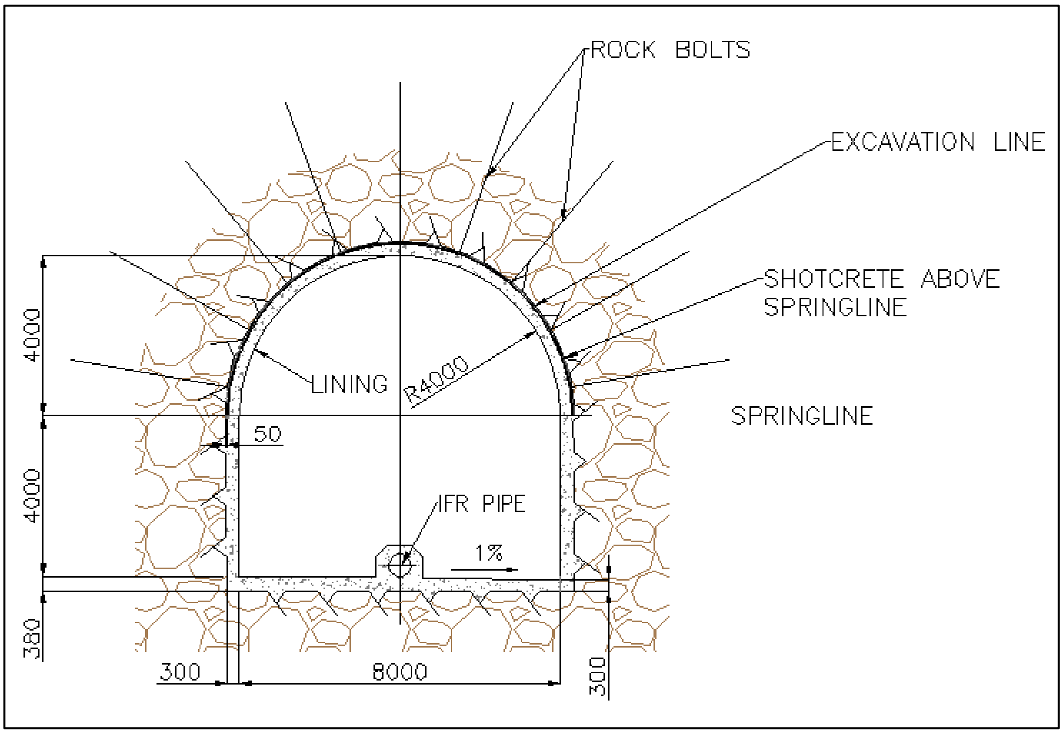

Figure 3.15: Tunnel 2 cross-section

A longitudinal section of the intake section of Tunnel 2 and the valve chamber is depicted in **Figure 3.16**, which also shows a section through the intake tower at a higher elevation.

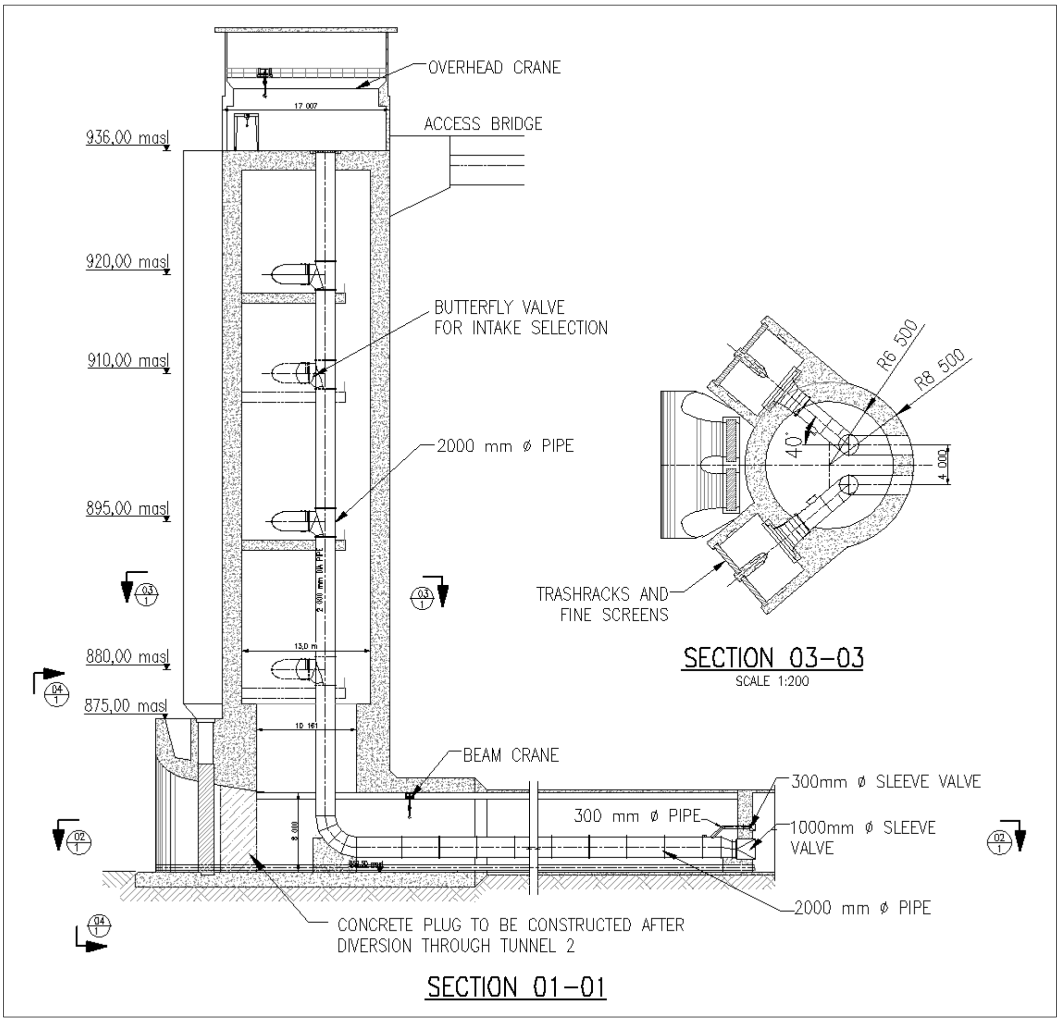

Figure 3.16: Section through intake structure

3.5.4 Cofferdam height determination using backwater analysis

Cofferdams will be constructed during different river diversion phases, as described in **Table 3.13**, to enable the construction of the river diversion tunnels and the river section of the main dam embankment. This section describes the backwater analysis conducted to determine the crest levels of the cofferdams.

The crest levels of Cofferdam 1 to 6 relate to the acceptable associated risk of delays and damages due to possible flood events. The crest levels were determined by means of empirical hydraulic calculations and backwater modelling (energy principle) utilizing tunnel discharge curves for the river diversion tunnels and HEC-RAS (Hydrologic Engineering Centres River Analysis System computer program).

a) Backwater calculations using the tunnel discharge curve

The use of the discharge curves for the determination of cofferdam crest levels was focussed on river diversion phases where water will be diverted through the tunnels only. This applied to the crest level determination, preventing overtopping, for Cofferdam 3, 4 and 5.

The assumption was made that the water surface level anywhere on the upstream side of the tunnel inlets will be the same as that of the headwater level at the tunnel inlets.

For conditions where the tunnel intakes are unsubmerged, the headwater level associated with a specific discharge through one of the 8 m diameter tunnels was calculated using the Manning equation for free flow conditions:

$$Q_{free\ flow} = \frac{1}{n}\,\frac{\left(A^{\frac{5}{3}}\right)}{\left(P^{\frac{2}{3}}\right)}\,S^{\frac{1}{2}} \qquad \textbf{(Equation 3-3)}$$

Where:

n	=	Manning's n-value (Used as 0.012 for this application)
A	=	Flow area (m²)
P	=	Wetted perimeter (m)
S	=	Slope of the tunnel (m/m)

For the conditions where the inlet and the outlet of the tunnel is submerged, and hence pressurised flow conditions apply, the following formula was used to determine the discharge with associated headwater:

$$Q = \sqrt{\frac{RL - TWL}{\left(1 + \Sigma K_s \Big/ 2gA^2\right) + \left(L \Big/ K^2 R^{1.33} A^2\right)}} \qquad \textbf{(Equation 3-4)}$$

Where:

RL	=	Reservoir level (masl)
TWL	=	Tail-water level (masl)

ΣK_s = Sum of singularity losses (m)

g = Gravitation acceleration (m/s^2)

A = Area of water flow (m^2)

L = Tunnel length (m)

R = Hydraulic radius (m)

Table 3.20 provides the stage/discharge relationship for each of the two tunnels as well as the discharge capacity through both tunnels simultaneously for different headwater levels. The discharge curves of a single tunnel and the two tunnels are shown in **Figure 3.17**.

Table 3.20: Discharge capacity of tunnels

Headwater Elevation (masl)	Discharge (m^3/s)		
	Tunnel 1	Tunnel 2	Total
859	0	0	0
860	58	58	116
861	164	164	327
862	290	290	580
863	429	429	857
877	679	679	1 359
880	744	744	1 487
885	841	841	1 681
890	928	928	1 857
895	1 009	1 009	2 018
900	1 084	1 084	2 167

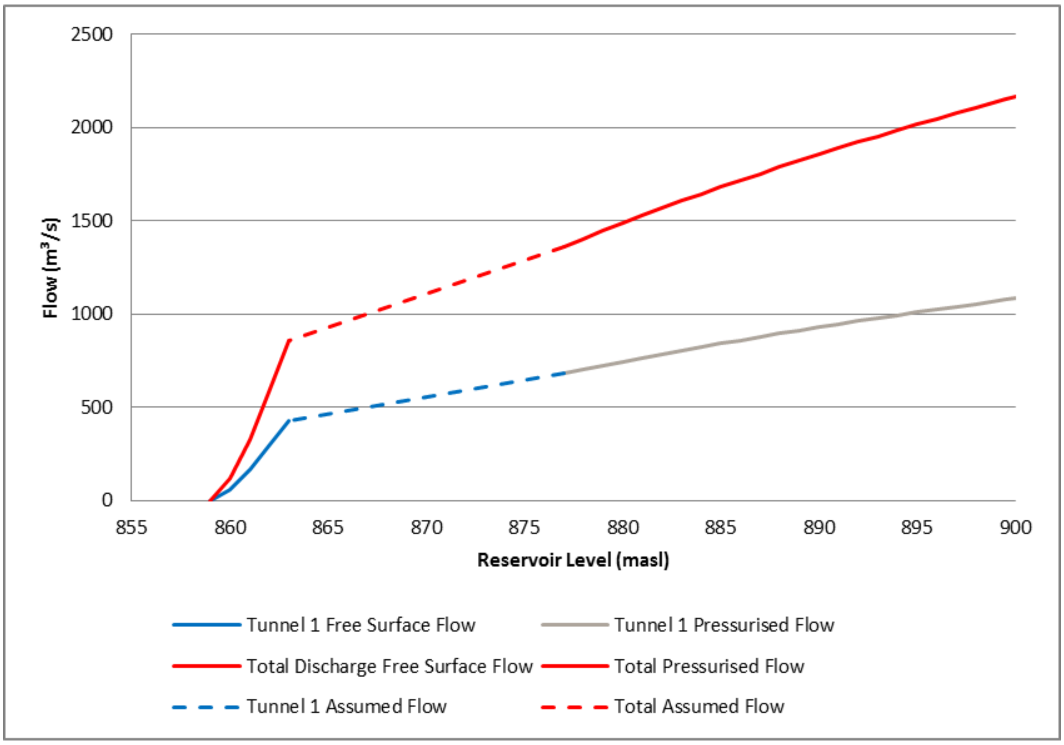

Figure 3.17: Discharge capacities for river diversion tunnels at Smithfield Dam

b) Backwater modelling using HEC-RAS

The computer program HEC-RAS was used to model the section of the uMkhomazi River relevant for the river diversion in order to determine the crest levels of all cofferdams.

The basic computational procedure of HEC-RAS, in this case of steady flow water surface profiles, is based on the solution of the one-dimensional energy equation:

$$Z_2 + Y_2 + \frac{a_2 {V_2}^2}{2g} = Z_1 + Y_1 + \frac{a_1 {V_1}^2}{2g} + h_e \qquad \textbf{(Equation 3-5)}$$

Where:

Z_1, Z_2	=	Elevation of the main channel inverts
Y_1, Y_2	=	Depth of water at cross-sections
V_1, V_2	=	Average velocities
a_1, a_2	=	Velocity weighting coefficient
g	=	Gravitational acceleration
h_e	=	Energy head loss

Energy losses are evaluated by friction (Manning's equation) and contraction/expansion. In situations where the water surface profile is rapidly varied, such as when water passes through critical depth, the energy equation is not applicable and the momentum equation is utilised. The water surface elevation at a cross-section is determined by an iterative solution of the energy equation or the momentum equation.

Cross-sections were taken along the applicable uMkhomazi River section and modified accordingly to include the infrastructure for each river diversion phase. The layout of the cross-sections is included as **Figure 3.D.1** in **Annexure 3 D**.

HEC-RAS only has the capability to model one-dimensional flow and is therefore unable to model the branching and confluence of water through the diversion tunnels to the river. For river diversion phases where water is diverted through the tunnels only, an appropriate assumed culvert system was placed in the river section reaching from the location of Cofferdam 4 to the location of the tunnel outlets in order to mimic the flow through the diversion tunnels.

HEC-RAS utilizes standard culvert hydraulics as set out in the Federal Highway Administration's (FHWA) *Hydraulic Design of Highway Culverts (1985)* to evaluate the flow condition, headwater depth, tailwater depth and the flow depth within a culvert.

c) *Comparison of backwater calculation and modelling results and recommendations*

Annexure 3 D includes a full description of the HEC-RAS methodology followed regarding the modelling of the different river diversion phases and the crest level results obtained. Also included are the crest levels determined with the tunnel discharge curves and how these results compare to that of the HEC-RAS results.

The crest level results from the discharge curve empirical calculations and the HEC-RAS modelling differed to a minor extent for some cofferdams. In such cases the higher crest level was considered. Results indicated in **Table 3.21** are as per the relevant structure.

3.5.5 Cofferdam characteristics

All Cofferdams, unless otherwise specified, are to be constructed with impervious earthfill material with large rip-rap on the sides to prevent extensive erosion damage due to the forces of the flowing water.

a) Cofferdam 1

Cofferdam 1 will be constructed at the tunnel intakes to allow for the excavation, lining, rock support etc. of the two tunnels during the first summer.

The 1:10 year recurrence interval flood water head at the proposed tunnel intakes was determined at level 861.89 masl. Cofferdam 1's crest must be constructed at level 862.4 masl to allow 0.5 m of freeboard. The height of Cofferdam 1 from the NGL will thus be 2.9 m. A cross-section of Cofferdam 1 is shown in **Figure 3.18**.

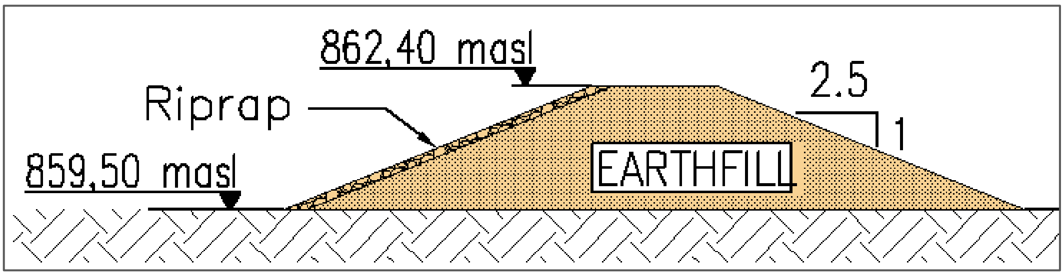

Figure 3.18: Cofferdam 1

b) Cofferdam 2

Cofferdam 2 will be constructed at the tunnel outlets to allow for the construction of the two 8 m wide tunnels during the first summer.

The HEC-RAS model water head at the proposed tunnel outlets, for the 1:10 year recurrence interval, is at level 859.63 masl. Cofferdam 2's crest must be constructed at level 860.1 masl which includes 0.5 m of freeboard. The height of Cofferdam 2, with reference to the NGL, will thus be 4.6 m. A cross-section of Cofferdam 2 is shown in **Figure 3.19**.

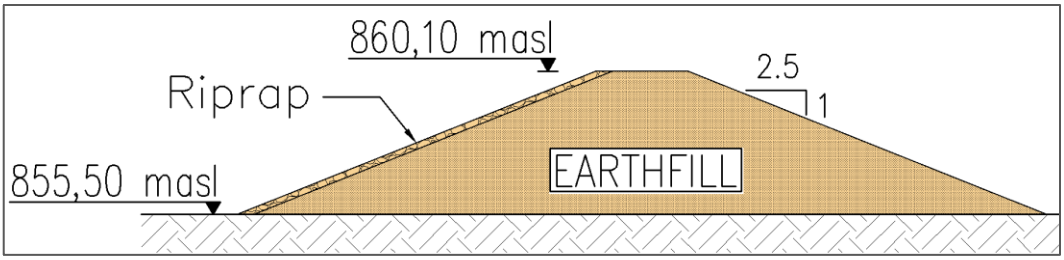

Figure 3.19: Cofferdam 2

c) Cofferdam 3

Cofferdam 3 will be constructed just downstream of the tunnel intakes to prevent water ingress into the main dam embankment area during the construction of Cofferdam 5 in the first winter.

Cofferdam 3 must accommodate the 1:10 year winter flood without overtopping in order to divert water through the tunnels only.

Results obtained from the tunnel discharge curve and the HEC-RAS model recommended that the crest, including 0.5 m freeboard, should be at 862 masl or 4 m high from the river bed level. A cross-section of Cofferdam 3 is shown in **Figure 3.20**.

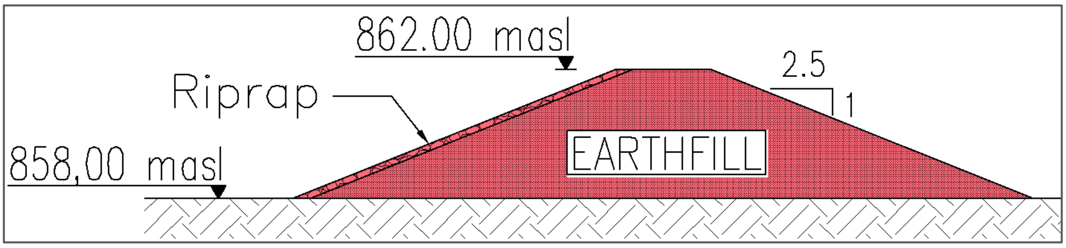

Figure 3.20: Cofferdam 3

Smaller cofferdams are required for the construction of Cofferdam 3, however the design thereof will only be included in the detail design.

d) Cofferdam 4

Cofferdam 4 will be part of the downstream toe of the main dam embankment to prevent back flow, released by the diversion tunnels, to enter the construction area of the main dam embankment in the uMkhomazi River section during the third summer.

The cofferdam will be an earth core rockfill dam. A drain pipe should be provided (with valve) through the dam for draining the rockfill of the main embankment.

The crest level of Cofferdam 4 is recommended to be at 862.3 masl or 6.3 m high to accommodate the 1:50 year flood event without overtopping. The height allows for 0.5 m of freeboard.

e) *Cofferdam 5*

Cofferdam 5 (**Figure 3.21**) must accommodate the 1:20 year winter flood event without overtopping to allow for the construction of Cofferdam 6 during the second winter.

This cofferdam will be a 7 m high roller compacted concrete (RCC) dam with the overflow at 863 masl for flood events larger than the 1:20 year winter flood. Cofferdam 5 will also form the upstream toe of the main dam embankment.

f) *Cofferdam 6*

Cofferdam 6 (**Figure 3.21**) will be part of the upstream part of the main dam embankment to allow for the construction of the main dam embankment in the uMkhomazi River section during the third summer.

Cofferdam 6 will be a rockfill section abutting the rockfill of the main dam embankment. The sealing of the cofferdam will be achieved with an HDPE membrane. This membrane will be anchored to the concrete gravity wall at the toe and against a concrete plinth up to level 888 masl. Allowance must be made for the differential settlement of the rockfill by specific design of the anchoring of the HDPE to the concrete wall and plinth.

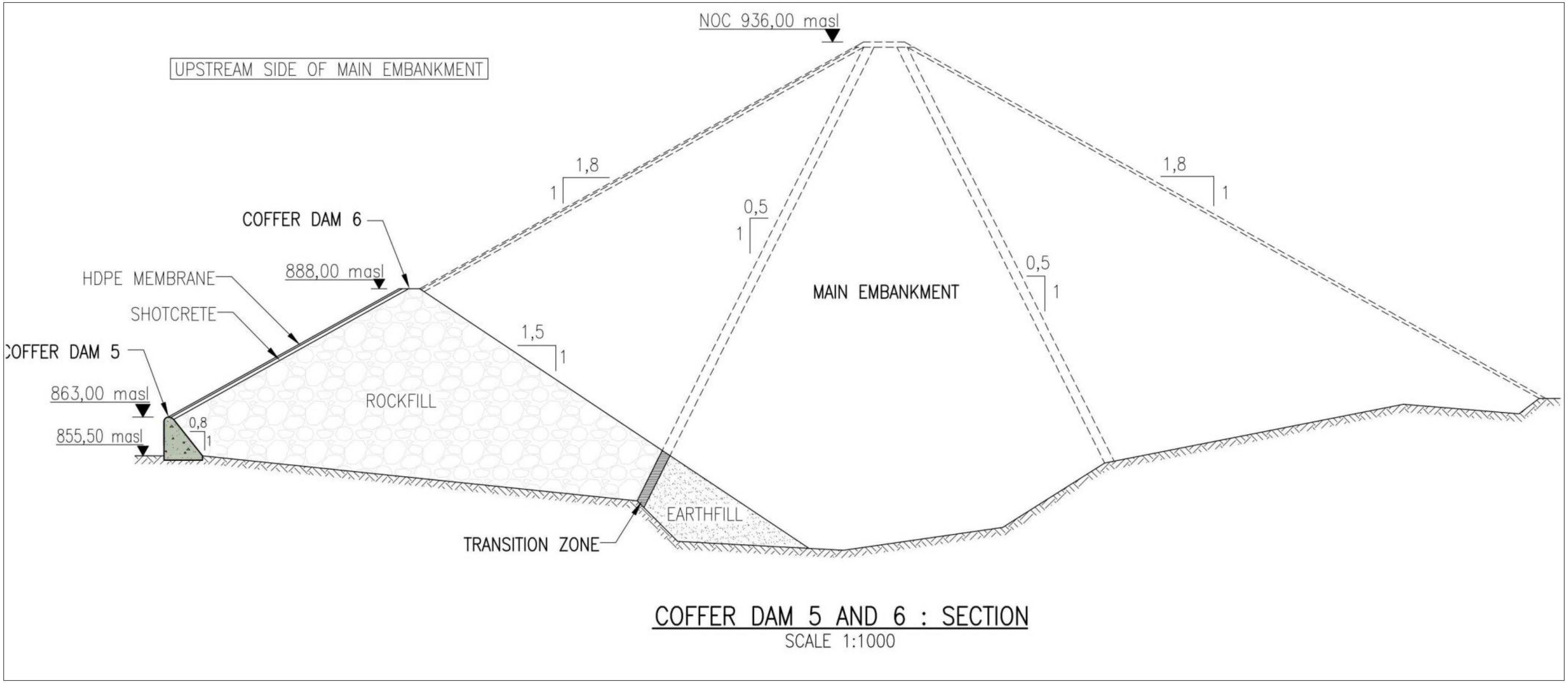

Figure 3.21: Layout of Cofferdams 5 and 6

g) Summary of cofferdam characteristics

Table 3.21 provides a summary of the crest levels of the various cofferdams and the flood event which they can accommodate.

Table 3.21: Cofferdam characteristics

Coffer dam	NGL (masl)	Accommodated flood event	Flood volume (m^3/s)	Crest elevation (masl)	Height from NGL (m)
1	859.5	1:10	937.0	862.4	2.9
2	855.5	1:10	937.0	860.1	4.6
3	858.0	1:10 winter	74.5	862.0	4.0
4	856.0	1:50	1 708.0	862.3	6.3
5	856.0	1:20 winter	145.1	863.0	7.0
6	856.0	1:50	1 708.0	888.0	32.0

3.5.6 Construction program

The full construction programme is described and included in **Section 15** of this report.

3.6 SPILLWAY DESIGN

3.6.1 General

The spillway options investigated are for one main spillway and a main spillway combined with a fuse plug spillway on the left bank of the saddle embankment dam.

The main spillway is a side channel spillway type consisting of an excavated approach channel to accommodate smooth flow lines for the improvement of the discharge coefficient, a concrete gravity ogee structure, a side channel and a return chute.

The fuse plug spillway consists of a concrete broad crested weir at FSL (930 masl) covered by non-cohesive material. Pilot channels are provided on the 1:200 year head water level, which will result in the breaching of the fuse plug.

The sizing of the spillway was based on flood attenuation studies and the analysis used the following information:

- Flood hydrographs as described in **Section 3.2.4**;
- Stage-storage volume and surface area curve as in **Section 3.3**; and
- Spillway discharge table in **Section 3.6.3**.

Freeboard requirements as determined in **Section 3.6.4** are compared to the SEF occurrence with the requirement that the embankment is not overtopped.

3.6.2 Spillway discharge formula

The discharge for the main and fuse plug spillway, according to the USBR's *Design of Small Dams* (1987), is expressed as:

$$Q = C\ L_{eff}\ H_e^{1.5} \qquad \textbf{(Equation 3-6)}$$

Where:

Q	=	Discharge (m³/s)
C	=	Discharge coefficient
L_{eff}	=	Effective length of the spillway (m)
H_e	=	Upstream head above the FSL, including the velocity head (m)

The discharge coefficient formula for an ogee spillway can be expressed as:

$$C = 1.587 + 0.593\left(\frac{H_e}{H_o}\right)^{0.5} \qquad \textbf{(Equation 3-7)}$$

Where:

H_o	=	Design head (Design flood = 1:200 year flood)

The discharge coefficient for the broad crested weir was taken as 1.7.

The effective length of the spillway (L_{eff}), considering that it was only affected by the two end abutments, was determined as:

$$L_{eff} = L - 2K_a H_e = L - 0.2H_e$$ **(Equation 3-8)**

Where:

L = Net length of the spillway (m)

K_a = Abutment contraction coefficient (0.10).

3.6.3 Spillway discharge tables and curves

The spillway discharge tables were determined for the following spillway options and are shown in **Table 3.22**:

- One main spillway with lengths of :
  - 120 m (Option 1);
  - 160 m (Option 2); and
  - 200 m (Option 3).
- Main spillway combined with fuse plug spillway with lengths of:
  - Main = 100 m, Fuse plug = 100 m (Option 4); and
  - Main = 150 m and fuse plug = 100 m (Option 5).

Table 3.22: Discharge tables for various spillway options

H (masl)	Q (m³/s)				
	Option 1	Option 2	Option 3	Option 4	Option 5
930.0	0.00	0.000	0.00	0.00	0.00
930.5	75.50	101.80	128.44	61.31	93.38
931.0	222.99	301.91	382.18	180.59	276.60
931.5	422.89	574.23	728.62	341.80	525.65
932.0	668.12	909.39	1156.08	539.14	831.89
932.5	954.58	1301.93	1657.75	769.23	1190.29
933.0	1279.47	1748.15	2229.02	1029.78	1597.44
933.5	1640.71	2245.30	2866.48	1319.08	2050.81
934.0	2036.68	2791.24	3567.48	2984.90	2548.43
934.5	2466.06	3384.23	4329.9	3587.00	4696.89
935.0	2927.75	4022.84	5151.82	4701.31	6182.92

H (masl)	Q (m^3/s)				
	Option 1	Option 2	Option 3	Option 4	Option 5
935.5	3420.81	4705.81	6031.84	4908.92	6460.63
936.0	3944.41	5432.09	6968.60	5625.76	7421.39
936.5	4497.83	6200.72	7960.93	6378.18	8432.69
937.0	5080.44	-	-	-	-
937.5	5691.65	-	-	-	-

3.6.4 Freeboard

The required freeboard based on the RDF was determined according to the *SANCOLD Interim Guidelines on Freeboard for Dams* (South African National Committee on Large Dams, 1990).

These guidelines indicate that the following combinations be considered for a large dam with a high hazard rating:

- **Combination 1:** Sum of the levels for the routed Recommended Design Flood (RDF) (1:200 year), the wind wave run-up for a 1:25 year event and the wind set-up.
- **Combination 2:** Sum of the levels for the routed RDF (1:200 year), the wind wave run-up for a 1:25 year event, the wind set-up and the flood surges and seiches.
- **Combination 3:** Sum of the levels for the 1:20 year flood, the wind wave run-up for a 1:100 year event, the wind set-up and flood surges and seiches.
- **Combination 4:** Wave height due to an earthquake, alone, was not investigated due to the low seismic horizontal acceleration for the Smithfield Dam site.
- **Combination 5:** Sum of the levels for routed RDF and wave run-up height due to a landslide.
- **Combination 6:** As no flood outlets are foreseen, this combination was not investigated.

The graphical presentation for determining the wind setup used in determining the minimum freeboard is included in **Annexure 3 E** as **Figure 3.E.1** and **Figure 3.E.2**.

The flood surges and seiches are taken as 1 m for large dams.

The wave run-up height as a result of a landslide was determined through a detailed study involving the geotechnical analysis of the slopes surrounding the reservoir and the calculation of wave heights based on potential slope failures. This study has been discussed in detail in **Section 3.15** and wave run-up height values associated with various potential landslides are presented in **Table 3.43**.

The results of the above-mentioned combinations are summarised in **Table 3.23** and this table indicates that combination 2 requires the largest freeboard, which will be the minimum freeboard, for all the spillway sizes.

Table 3.23: Summary of the determination of freeboard for various spillway size combinations

Combi-nation	Routed RDF (height above FSL m)	20-year flood	Wind wave and Run-up		Wind set-up	Flood surges and seiches	Land-slide Wave*	TOTAL (m)
			25-year event	100-year event				
Spillway length = 200 m								
1	3.11		0.67		0.012			**3.79**
2	3.11	-	0.67	-	0.012	1	-	**4.79**
3		2.09	-	0.70	0.012	1	-	**3.81**
5	3.11	-	-	-	-	-	1.41	**4.52**
Spillway length = 160 m								
1	3.53		0.67		0.012			**4.21**
2	3.53	-	0.67	-	0.012	1	-	**5.21**
3		2.25	-	0.70	0.012	1	-	**3.96**
5	3.53	-	-	-	-	-	1.41	**4.94**
Spillway length = 150 m								
1	3.70		0.67		0.012			**4.38**
2	3.70	-	0.67	-	0.012	1	-	**5.38**
3		2.55	-	0.70	0.012	1	-	**4.26**
5	3.70	-	-	-	-	-	1.41	**5.11**
Spillway length = 120 m								
1	4.13		0.67		0.012			**4.81**
2	4.13	-	0.67	-	0.012	1	-	**5.81**
3		2.95	-	0.70	0.012	1	-	**4.66**
5	4.13	-	-	-	-	-	1.41	**5.54**
Spillway length = 100 m								
1	4.57		0.67		0.012			**5.25**
2	4.57	-	0.67	-	0.012	1	-	**6.25**
3		3.33	-	0.70	0.012	1	-	**5.05**
5	4.57	-	-	-	-	-	1.41	**5.98**

**Obtained from the "Geotechnical Report" P WMA 11/U10/00/3312/3/1*

3.6.5 Option 1: One main side channel spillway

a) General

Three spillway lengths (ogee type) of 120 m, 160 m and 200 m were investigated for the SEF ($RMF_{+\Delta}$).

b) Flood routing

The discharge tables for various ogee crest lengths were determined and used in the flood routing programme (FLOOD2) to determine the routed headwaters and subsequently the non-overspill crest (NOC) level for the SEF for the dam.

The flood routing results for the three options are included in **Annexure 3 E** as **Table 3.E.1** and are summarised in **Table 3.24**.

Table 3.24: Summary of the routed SEF and minimum required freeboard for one main overspill structure

Routed SEF information				Min required freeboard associated with interim guidelines*
Spillway length (m)	Routed SEF (5 650 m^3/s)	Non overspill level (masl)	Freeboard (m)	
120	4 960	936.90	6.90	5.81
160	5 200	935.85	5.85	5.21
200	5 325	935.15	5.15	4.79

**SANCOLD Interim guidelines on freeboards for Dams (refer to Table 3.23)*

The freeboard related to the SEF for only the main spillway is more than the required minimum freeboard determined in **Section 3.6.4**. Thus, the NOC level related to the routed SEF will be applicable.

c) Preliminary cost estimate

The preliminary cost estimates for the three options were determined using the cost model developed during the dam type selection phase and are summarised in **Table 3.25**.

Table 3.25: Summary of the preliminary cost estimate for various main spillway ogee crest lengths

Activity	Spillway length		
	L = 120 m	L = 160 m	L = 200 m
NOC (masl)	936.90	935.85	935.15
Cost: Earth core rockfill embankment (R)	565 164 649	545 191 015	532 090 120
Cost: Diversion works (R)	55 447 659	55 447 659	55 447 659
Cost: Spillway and chute (R)	94 846 345	101 196 160	108 659 490
Cost: Intake and outlet works (R)	77 063 316	76 662 808	76 578 230
Cost: Saddle embankment (R)	160 205 886	148 956 479	141 045 569
Cost: Total (R)*	**952 727 855**	**927 456 124**	**913 821 068**

**Excluding costs for preliminary and general, preliminary works, accommodation, contingencies, planning design, supervision, relocation, land acquisition and VAT.*

From **Table 3.25** it is clear that for a longer spillway length and a lower NOC level, the estimated activity costs are less and therefore favoured.

The breakdown of the preliminary cost estimates is included in **Annexure 3 E** as **Table 3.E.2**.

3.6.6 Option 2: Main side channel spillway combined with a fuse plug spillway

a) General

The following two options for a main spillway and fuse plug were investigated for the SEF ($RMF_{+\Delta}$):

- Main spillway length of 150 m and fuse plug length of 100 m; and
- Main spillway length of 100 m and fuse plug length of 100 m.

b) Flood routing

The discharge table for various spillway lengths were determined and used in the flood routing programme (FLOOD2) to determine the routed NOC head water level for the dam. The flood routing results for the two options are included in **Annexure 3 E** as **Table 3.E.4** and are summarised in **Table 3.27**.

The operation rule followed included that the main spillway must be able to discharge the 1:200 year flood before the fuse plug is activated by pilot channels on the 1:200 year flood level.

The flood level for the 1:200 year flood only discharging at the main spillway was determined by flood routing of the 1:200 year flood hydrograph for the two main spillway lengths, and is summarised in **Table 3.26**.

Table 3.26: Summary of the routed 200 year flood over the one main side channel spillway

Main Spillway length (m)	Routed 200 year flood (2 620 m³/s)	200 year flood level (masl)	Head water (m)
100	2 030.71	934.57	4.57
150	2 249.52	933.70	3.70

The routed SEF and hence the NOC level for the various spillway and fuse plug lengths were determined by routing the SEF using the FLOOD2 computer programme. The flood routing results for the two options are summarised in **Table 3.27**.

Table 3.27: Summary of the routed SEF for a main overspill structure and fuse plug

Spillway length (m)	Fuse plug length (m)	Required 1:200 year water level (m)	Routed SEF (5 650 m³/s) (m³/s)	Routed non overspill level (masl)	Routed SEF Freeboard (m)	Min Freeboard (m)
100	100	4.57	5 249	935.74	5.74	5.81
150	100	3.70	5 415	934.91	4.91	5.38

From **Table 3.27** the following can be derived. The 1:200 year flood level is less than the routed SEF freeboard required. The freeboard for the routed SEF is less than the required minimum freeboard as determined in **Section 3.6.3**. For both these options the NOC will be set to the rounded up value for the routed SEF and a parapet wall will be added for the additional freeboard required. This will result in the dam levels as described in **Table 3.28**.

Table 3.28: Dam spillway options and the minimum required freeboard

Routed SEF information						Min required freeboard associated with interim guidelines*
Spillway length (m)	Fuse plug length (m)	Non overspill level (masl)	Freeboard (m)	Parapet wall height (m)	Total freeboard (m)	
100	100	935.80	5.80	0.20	6.00	5.81
150	100	935.00	5.00	0.40	5.40	5.38

**SANCOLD Interim guidelines on freeboard for dams*

c) Preliminary cost estimate

The preliminary cost estimates for the two options for the main spillway with a fuse plug spillway were determined using the cost model developed during the dam type selection phase and are summarised in **Table 3.29**. The additional cost of an access bridge and parapet wall were included in the cost estimate, as these costs were not initially included in the developed cost model.

Table 3.29: Summary of the preliminary cost estimate for various options of a main side channel spillway with a fuse plug spillway

Activity	Cost (R) for Spillway length (m)	
	Main L = 100 Fuse plug L = 100	Main L = 150 Fuse plug L = 100
Earth core rockfill embankment	544 249 598	529 304 907
Diversion works	55 447 659	55 447 659
Spillway and chute	70 642 545	76 566 817
Intake and outlet works	77 063 316	76 547 165
Saddle embankment*	142 527 582	133 532 781
Fuse plug spillway	23 046 500	23 865 561
Total (R)**	**912 977 200**	**895 264 890**

* *Excluding the section replaced by the fuse plug spillway*

** *Excluding of cost for Preliminary and General, Preliminary works, accommodation, contingencies, planning design, supervision, relocation, land acquisition and VAT.*

From **Table 3.29** it is clear that estimated activity costs for the main side channel spillway of 150 m with a fuse plug spillway of 100 m are less and therefore favoured.

The breakdown of the preliminary cost estimates is included in **Annexure 3 E** as **Table 3.E.3**.

3.6.7 Estimated cost comparison of the spillway options

The estimated cost determined for the various spillway options are compared and summarised in **Table 3.30**.

Table 3.30: Summary of the estimated cost comparison for various spillway configurations

Spillway length (m)	Fuse plug length (m)	NOC (masl)	Estimated Activity Cost (R)*
One main side channel spillway			
120	N/A	936.90	952 727 855
160	N/A	935.85	927 456 124
200	N/A	935.15	913 821 068
Main side channel spillway with fuse plug spillway			
100	100	936.00 (935.80**)	912 977 200
150	100	935.40 (935.00**)	895 264 890

* *Exclude cost for Preliminary and General, Preliminary works, accommodation, contingencies, planning design, supervision, relocation, land acquisition and VAT*

** *Embankment NOC without parapet wall*

The estimated costs of a main spillway length of 200 m and a main side channel spillway of 100 m with fuse plug spillway of 100 m compare well, while the option with a main side channel spillway of 150 m with fuse plug spillway of 100 m has the lowest cost and was selected as the preferred spillway configuration.

3.6.8 Recommendation on spillway configuration

Additional geotechnical investigations are required in the tender design phase to determine foundation conditions for the position of the main spillway as well as the erosion potential at the fuse plug spillway. Subsequently a total cost optimisation of the dam freeboard and spillway width should be carried out.

However, a main side channel spillway with a crest length of 150 m and a 100 m wide fuse plug spillway were selected as the preferred spillway option for the feasibility design phase as this is the configuration with the lowest cost.

3.6.9 Layout for feasibility design

For the recommended spillway configuration the PMF was routed to determine the layout for the feasibility design. The flood routing of the PMF (6 185 m^3/s) resulted in a routed flood of 5 910 m^3/s and a non-overspill level of 935.2 masl. The minimum required freeboard is at level 935.38 masl. An additional 600 mm was added to the determined non-overspill crest to compensate for possible future settlement of the dam crest and increased floods due to climate change. The NOC level was selected at 936 masl for the feasibility design.

The layout of the spillways is shown in **Figure 3.E.3** and the fuse plug design in **Figure 3.E.4**, included in **Annexure 3 E**.

3.6.10 Overspill structure

The overspill structure is designed for an ogee shape. The *SANCOLD guidelines* require that for a Category III dam the ogee must be designed for a RDF of 1:200 year (2 620 m^3/s). The level of the approach channel is at 926 masl, resulting in the pool depth to be equal to the design head. The discharge table, taking into account the excavation level of the approach channel, was used to determine the headwater height of the RDF and hence used to define the shape of the ogee structure.

The calculations for the ogee shape are based on the Waterways Experiment Station (WES) standard spillway shapes (*Chow, 1959*). The formulae which apply are graphically presented in **Figure 3.22** and the determined values in **Table 3.31**.

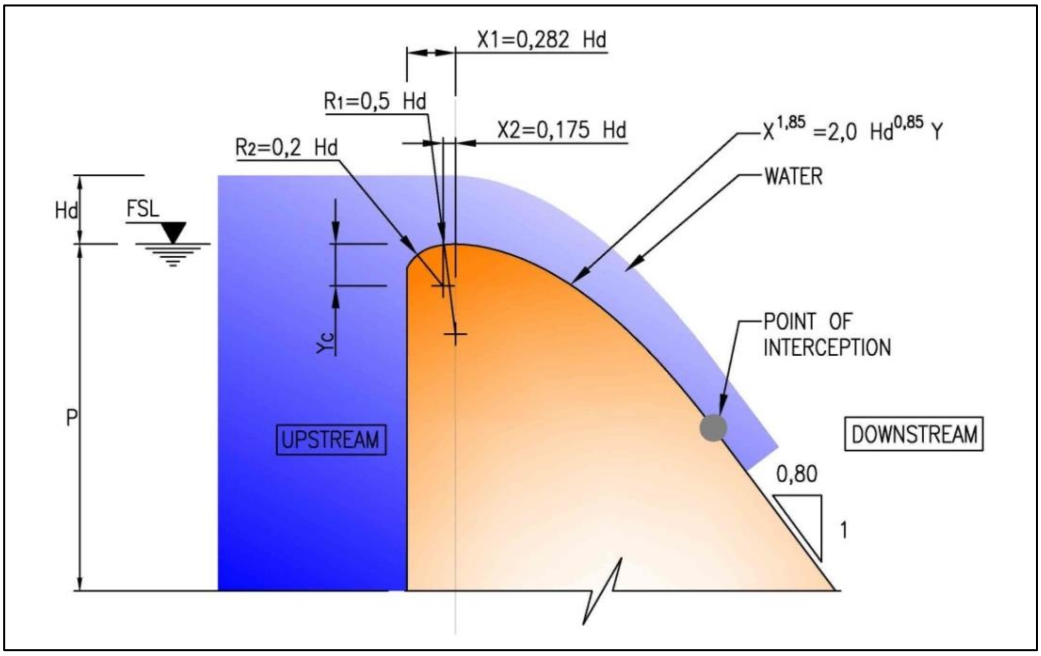

Figure 3.22: Graphical presentation of ogee shape

Table 3.31: Ogee shape characteristics

Parameter	Value / formula
H_d	4.07 m
R_1	2.035 m
R_2	0.814 m
X_1	1.148 m
X_2	0.712 m
Curve shape	$Y = 0.15164 X^{1.85}$
Point of interception	X = 5.801; Y = 3.919

3.6.11 Chute

The chute will be excavated to the rock level with side slope of 1V:1H. The width of the chute is determined using the assumption that 100 m³/s is required per width of chute. A flow of 3 700 m³/s will be discharged through the main side channel spillway when an SEF occurs and the fuse plug spillway is breached, resulting in an approximate chute width of 37 m. The width of the chute was taken as 40 m.

The chute will be lined with a layer of concrete with a minimum thickness of 500 mm and anchored into the rock with anchor bars drilled and grouted into the rock. The concrete lining will be up to the water level for the partly discharged SEF (3 700 m³/s) in the chute as determined in HEC-RAS, from where the ground will be shaped to 1V:1.5H until it reaches the natural ground level. The chute and overspill is shown in **Annexure 3 E** in **Figure 3.E.5 (Vol 2, p 65)**.

The concrete lining also needs to be protected by a comprehensive system of drains underneath it with collector drains on each side of the chute. The anchors need to hold down the concrete lining against a possible full static uplift water pressure, and the surface finish of the lining must be of a high standard to minimise the effects of cavitation should high velocity flows occur. The anchors and drainage system should be designed in the detail design phase.

The chute will end with a ski jump discharging into a plunge pool. The ski jump for Impofu Dam was adopted for the feasibility design and will have to be optimised during the tender design phase. Refer to **Figure 3.E.6** in **Annexure 3 E** for detail on the ski jump.

The plunge pool will also serve as a quarry and the excavated material used to construct the dam.

3.7 ZONING OF DAM

3.7.1 Main dam

The principle of using available materials at lowest cost where possible was used for the zoning of the dam.

The material investigation showed two types of rockfill. The soft rockfill (moderately weathered to unweathered shale) is layered on top of the coarse rockfill (slightly weathered to unweathered dolerite), and needs to be removed before the coarse rockfill can be mined. The soft rockfill (3D material) can be used in the inner zones of the rockfill shells, adjacent to the central core of the dam, and the coarse rockfill (3C material) used in the outer shells of the dam embankment.

Table 3.32 shows the selected zones of the embankment and the proposed compaction and grading specifications. The layout and maximum cross-section of the main embankment is shown in **Annexure 3 F** as **Figure 3.F.1** and **Figure 3.F.2**, respectively.

Table 3.32: Compaction and grading specifications of selected zones of the embankment

Component	Zone	
	3D (soft material)	3C (coarse material)
Classification	Quarry run rockfill (moderately weathered to unweathered shale)	Quarry run rockfill (slightly weathered and unweathered dolerite)
Gradation	0.075 mm: maximum 10% <25 mm: maximum 50%	1 m maximum size
Lift height (m)	1.0	1.0
Type of roller	10 tonne vibratory roller	10 tonne vibratory roller
Passes	Min. 6	Min. 6

3.7.2 Saddle dam

The required material for the saddle dam consists of clayey sand, completely weathered and highly weathered material as well as sand for the filter zones. The clayey sand will be used in the core zone and the highly and completely weathered material in the shell zone of the dam. This material will be obtained from the excavation for the rockfill material. The sand for the filters will be imported from commercial sources.

The saddle wall cross-section is shown in **Annexure 3 F** as **Figure 3.F.3**.

3.8 AVAILABILITY AND QUALITY OF CONSTRUCTION SOIL AND ROCK MATERIALS

3.8.1 Background of material identification

The geotechnical investigation (*Smithfield Dam Construction Materials and Geotechnical Investigation (Geotechnical Report: Supporting Document 3 (P WMA 11/U10/00/3312/3/2/3)*) identified three borrow and four quarry areas as shown in **Annexure 3 C** as **Figure 3.C.1** and **Figure 3.C.2**. The type of materials, quality of the materials and their uses are described in **Table 3.33**.

Table 3.33: Type, quality and uses of soil and rock materials

Type of material / location	Quality	Use application
Overburden	Soil mixed with topsoil	Rehabilitation of disturbed areas
Impervious earthfill material in Borrow Areas A and B	Classify as CI and few CH in Casagrande Classification*. Two samples have a plasticity index (PI) of 30 to 40 and a liquid limit (LL) of 60 to 70. These materials must be mixed with materials with lower values for quicker construction.	Core zones of Smithfield Main and Saddle Dam Embankments
Semi-pervious earthfill materials – all over the site expect for Borrow Areas A and B	Classify as CL or CI in Casagrande Classification	To be used in zones of embankment
Soft shale rockfill	Moderately weathered shales	To be used in zones of embankment
Coarse shale rockfill below dolerites in Quarry I not shown in table.	Good coarse rockfill	To be used in zones of embankment dam if dolerite quantity is not sufficient
Weathered dolerites	Soil to be used as earthfill	May be usable in outer zones of embankment dams
Rockfill, aggregates, filters, transition zones, rip-rap	Moderately weathered dolerite	Rockfill in main dam embankment Rip-rap and transition zones in saddle dam embankment Transition zones in main dam embankment

Plot shown in **Annexure 3 G as **Figure 3.G.3**.*

The layout of Quarry I described in the *Geotechnical Investigation* was based on a rockfill saddle dam (steeper slopes) with its toe further away from the quarry. The *Dam Type Selection Report* has shown that an earthfill embankment dam for the saddle embankment is the most economical dam type, resulting in a layout of which the toe is closer to the proposed quarry. Also, the good quality rockfill location is confined by a fault/displacement on the western side of it.

Further layouts of the quarry were considered in this report, as described in the following section.

3.8.2 Update from previous information

Further material allocation of Quarry I was executed by 3D modelling (AutoCAD Civil 3D), and the available dolerite and shale quantities were determined for two scenarios as follows:

- **Scenario 1**: The confined area with the unweathered dolerite between the saddle dam (but 40 m away) and the displaced area on the east.

- **Scenario 2**: The full area investigated. Note: to the eastern side of the displaced area the dolerites are of varying quality and may not be unweathered (uncertain distribution of quality).

The material balances for the two scenarios are shown in **Table 3.34**. The total available quantities indicated include a bulking factor of 1.27 for shale and 1.29 for dolerite. A plan layout of the quarry with the cross sections and proposed development of the quarry is indicated in **Annexure 3G** as **Figure 3.G.1** and **Figure 3.G.2**.

3.8.3 Conclusions

From **Table 3.34** it is clear that sufficient materials are available. If the dolerites are insufficient, the unweathered shale below the dolerites can be used in inner zones of the rockfill embankment, thereby saving on dolerite. Material to the eastern side of the displacement, in Quarry I, could be utilised if at all required.

It is therefore proposed that final design of the layout is based on a main ECRD with an earthfill saddle dam during the tender design phase, unless substantial information to contradict this is obtained. If necessary, further drilling investigations should also be done during this phase.

3.9 CREST WIDTH

A crest width was selected to allow vehicle access and limit the size of the embankment. A 7 m section between the guardrails is sufficient for vehicles to access the crest. The recommended crest width is thus 8 m to allow for the positioning of the guardrails.

Table 3.34: Required materials versus available material

Material use	Volume (m³)						
	A	B	C	D	E	F	G
	Overburden for soil: Organic topsoil	Clayey sand transported surface material	Completely & highly weathered shales	Unweathered to moderately weathered shales	Highly & moderately weathered dolerite	Slightly weathered & unweathered dolerite	Sand
River diversion	0	8 512	0	0	0	530	0
Main embankment	0	608 645		572 389	206 875	3 772 300	0
Saddle Embankment	0	152 891	855 286	0	0	141 412	19 211
Tunnel	0	0	0	0	0	39 208	53 000
Dam outlet works, Tunnel inlet, spillway and fuse plug	0	8 512	12 370	0	0	79 316	87 186
Total required [(1)]	**0**	**778 500**	**867 600**	**572 300**	**206 800**	**4 032 700**	**159 300**
Borrow area A	120 000	800 000	0	0	50 000	0	0
Borrow area B	100 000	850 000	0	0	100 000	0	0
Borrow area C	0	0	0	0	0	0	0
Quarry I: Scenario 1 (Higher certainty of quality) [(9)]	46 632	18 653	772 030	772 030	176 931	2 277 477	0
Quarry II	40 000	200 000	170 000	44 000	850 000	720 000	0
Quarry III	20 000	25 000	20 000	10 000	815 000	123 000	0
Quarry IV	5 000	7 000	110 000	13 500	0	0	0
Excavation main wall	230 371	1 027 138	515 221	17 669	18 326	15 255	0
Excavation saddle wall	23 962	29 915	59 2821	32 937	0	0	0
Total available on site [(2)]	**585 965**	**1 930 568**	**2 180 072**	**890 136**	**2 010 257**	**3 135 732**	**0**
Imported [(3)]	0	0	0	0	0	0	159 397
Total available with bulking factor	**585 900**	**1 930 500**	**2 768 600**	**1 130 400**	**2 613 300**	**4 076 400**	**159 300**

Material use	Volume (m³)						
	A	B	C	D	E	F	G
	Overburden for soil: Organic topsoil	Clayey sand transported surface material	Completely & highly weathered shales	Unweathered to moderately weathered shales	Highly & moderately weathered dolerite	Slightly weathered & unweathered dolerite	Sand
Stockpiled (4)	585 965	778 560	867 656	572 389	206 875	4 032 766	0
Spoiled (5)	0	0	0	0	2 406 459	0	0
Dam forming (6)	0	778 560	867 656	572 389	0	4 032 766	159 397
Surplus (7)	0	1 152 008	1 901 035	558 083	2 406 459	43 686	0
Pecentage remaining (%) (8)	-	60	69	49	-	1.1	-
Quarry I: Scenario 2 (Low certainty of quality) (9)	72 000	28 800	1 192 200	1 192 200	213 800	2 752 500	0

Notes:

*(1) The total volume of material **required** for the (i) main dam, (ii) saddle dam, and all additional infrastructure including the (iii) diversion works, (iv) intake structure, (v) spillway i.e. approach, chute and plunge pool, and (vi) outlet works.*

*(2) The total volume of material **available on site** from (i) the main dam excavation, (ii) the saddle dam excavation, (iii) Quarry I (left flank), (iv) Quarry II (plunge pool), (v) Quarry III (spillway approach), (vi) Quarry IV (tunnel inlet), (vii) Borrow area A, (viii) Borrow Area B and (ix) Borrow Area C.*

*(3) The total volume of material that has to be **imported from a commercial source**.*

*(4) The total volume of material that need to be **stockpiled** for later use.*

*(5) The total volume of material that need to be **spoiled** in the designated waste disposal site.*

*(6) The total volume of material that need to be **used in the forming of the specific dam type**.*

(7) The total volume of surplus materials that is kept undisturbed in the respective quarries or borrow areas if the excavated material is sufficient.

(8) Percentage of material that will remain in the quarries or borrow areas.

*(9) Material in Quarry I which must be further investigated for suitability before/during detail design. Refer to **Section 3.8.2** for detail.*

3.10 STABILITY OF DAM EMBANKMENT

Slope stability analyses were conducted with the tested parameters for the different soil types from the geotechnical investigations to determine the optimal slopes of each of the various dam types. Parameters used in this exercise are summarised in **Table 3.35**. The results from the soil stability analyses are included in **Annexure 3 H** as **Figure 3.H.1** to **Figure 3.H.8**, with the resultant slopes for the various dam types summarised in **Table 3.36**.

Table 3.35: Engineering properties for the various material types

Material No.	Material type	Phi – Φ (°)	Cohesion – C (kPa)	Density (kg/m³)
A	Overburden for soil: Organic topsoil	26	23	1 300
B	Clayey sand transported surface material	26	23	1 730
C	Completely and highly weathered shales	35	0	2 049
D	Unweathered to moderately weathered shales	38	0	2 100
E	Highly and moderately weathered dolerite	36	0	2 100
F	Slightly weathered and unweathered dolerite	40	0	2 200
-	Undisturbed dolerite	40	100	2 720
-	Concrete	35	500	2 300

Table 3.36: Resultant slopes for various dam types

Dam type	Upstream slope	Downstream slope
Zoned earthfill embankment dam	1(V):3(H)	1(V):2.5(H)
Earth core rockfill dam (ECRD)	1(V):1.8(H)	1(V):1.8(H)

3.11 SEEPAGE CONTROL

Seepage through the foundation will be controlled with a cement grout curtain drilled at the clay core position. The small amount of seepage passing through the core will be contained with filters immediately downstream of the core and prevents the seepage from carrying core material away.

Refer to **Section 3.4.2** for grout curtain depths as directed by the *Smithfield Dam: Materials and Geotechnical Report P WMA 11/U10/00/3312/3/2/3*.

3.12 FILTER/TRANSITION DESIGN

The filter/transition layer design is based on layers with the following thicknesses:

- Sand filter layer around the clay core: 2 m; and
- Gravel transition layer next to the sand filter layer: 2 x 1 m.

3.13 DAM OUTLET DESIGN

3.13.1 General arrangement

The outlet works will release water into the uMkhomazi River for environmental requirements and in the case of emergency drawdown conditions.

The outlet works are positioned with a circular intake tower on top of the intake section of the second river diversion tunnel and outlet valves further downstream. This tunnel will serve as a permanent outlet where the released water will be conveyed through the tunnel and exit into the uMkhomazi River. The general layout and the arrangement of pipes and valves are shown in **Figure 3.12** to **Figure 3.16**.

The pipe work in the intake tower consists of a twin or dual system comprising of multi-level intakes at different levels with butterfly valves for selecting the level at which water is to be drawn off, and sleeve valves in the downstream outlet valve chamber for controlling the release volumes.

The intakes will be protected with precast concrete trash racks and stainless steel fine screens to prevent blockage by floating debris. Emergency gates are required for closure at the bellmouth intakes for maintenance purposes.

A superstructure with overhead gantry crane on top of the intake tower enables the operation of the fine screens and emergency gates. A combination of cranes allows for valves to be transported for installation and maintenance purposes.

The outlet works, including the intake tower, can be accessed via a bridge from the main dam embankment. The connection between the access bridge and the intake tower should be designed during the detail design phase to limit the influence of seismic activity on the integrity of the structure.

3.13.2 Required outlet capacity

The outlet works is designed to:

- empty Smithfield Dam during emergency drawdown conditions; and
- release the ecological water requirements (EWR).

a) Emergency draw down

With reference to the *Design Criteria Memorandum* (TCTA, 2009), outlet works should be capable of lowering the reservoir level from FSL to half depth in 60 days and to the lowest drawdown level (LDL) within 120 days. The half depth of Smithfield Dam is considered to be at the approximate level of 895 masl.

Emergency drawdown is required when the water level in the dam must be reduced to ensure the safety of the dam.

b) Ecological water requirement (EWR)

The EWR downstream of Smithfield Dam was determined from the daily flows as measured at gauging weir U1H005. These daily flows were patched, naturalised and provision for catchment development to 2050 was modelled in these flows.

The target flows to be released from Smithfield Dam to meet the EWR are provided in **Table 3.37**. These flows are provided with an exceedance probability.

Table 3.37: EWR requirement

Exceedance Probability	EWR target (m^3/s)
100.00%	0.0
50.00%	3.6
20.00%	9.1
10.00%	15.9
5.00%	25.6
2.00%	43.0
1.00%	59.9
0.50%	83.7
0.10%	143.8
0.05%	152.6
0.00%	235.2

3.13.3 Minimum operating level (MOL)

The MOL for water to be abstracted through the uMkhomazi – uMlaza Tunnel is set at 887.2 masl. The same level is considered relevant for the EWR releases, however for draw down conditions it is required that the dam be drawn down to at least the LDL at level 880 masl.

3.13.4 Layout requirements

a) *Multi-level intakes*

To ensure the impact of the dam and its management on the downstream aquatic life is minimised, four intake levels are recommended.

The centre-line levels of the intakes as proposed in the *Water Quality and Limnological Report (P WMA 11/U10/00/33/3312/3/1)* are listed in **Table 3.38**.

Table 3.38: Outlet works intake levels

Intake level	Meters above sea level	Intervals (m)
L1	920	10
L2	910	10
L3	895	15
L4	880	15

**FSL at 930 masl*

Two outlet systems are provided with the intakes staggered between the two outlet systems. Each intake connects to one of the two vertical collector pipes which are extended to the top of the intake tower for aeration.

Due to temperature and stratification compliance, the first two intake levels will be used for the majority of the time for EWR releases. The bottom level intakes ensure that water can still be released down the river with the MOL at 887.2 masl without vortex formation. Two intakes are required at the bottom intake level to accommodate emergency drawdown conditions.

b) *Intake bays*

Two intake bays are provided for the dual system of outlet pipes. The intake bays will extend outward from the circular intake tower. The intake bays have been sized to ensure sufficient approach flow area at the entrance and hence maintain an acceptable flow velocity through the trashracks and fine screens.

c) Trashracks and fine screens

Trashracks and fines screens prevent floating trash and debris, mostly occurring near the water surface, to be drawn into the intakes and damage equipment.

The upstream end of the intake bays are protected by fixed coarse precast concrete trashracks followed by removable stainless steel fine screens downstream.

The fine screen panels are lowered into guides embedded in concrete piers. These removable fine screen panels are each fitted with a tray at the upstream bottom to collect trash or debris when the screens are hoisted for cleaning purposes.

A grappling beam for handling the screens will be provided with storage in a rack on the deck.

d) Emergency gates

Emergency gates, one for each intake bay, are required to close off the bellmouth intakes during emergencies and for maintenance purposes. Built-in parts and guides are provided for handling the gate and for sealing around any of the intake bellmouths.

e) Drywell

A circular shape was proposed for the intake tower as it provides more seismic resistance than a square or rectangular structure. Furthermore, compressive stresses rather than tensile stresses are induced on the structure, resulting in less concrete reinforcing being required.

The drywell houses the intake level selector valves (butterfly valves), provides access to the valves, and is used for installation, removal and replacement of the valves, if necessary.

The drywell can be accessed from the bridge linked between the tower and the embankment. A lift and staircase in the intake tower will be required for inspection and maintenance purposes of the structure and the valves.

f) Super structure and overhead cranes

An overhead gantry crane is required on the deck of the intake tower to handle the fine screens and emergency gates.

A combination of gantry cranes enables the installation and removal of valves and their equipment for refurbishment. A single beam crane runs along the top of the outlet tunnel for removal of the downstream sleeve valves and equipment. The overhead gantry crane on the deck of the intake tower allows for the valves to be removed and lowered onto a 10 ton truck.

3.13.5 Sizing of pipes and valves

The pipes and associated valves were sized to accommodate:

- The emergency drawdown of the dam within the required time; and
- The EWR releases.

a) Emergency drawdown requirements

As stated in **Section 3.13.2**, the water level of Smithfield Dam must be lowered from FSL to half the water depth in 60 days and to LDL within 120 days while the maximum flow velocity in the pipes is limited to 7 m/s to prevent excessive noise, vibration and wear, especially of the butterfly valves.

The following formulas were used to determine the flow through the outlet works and hence the draw down duration for a range of pipe and valve sizes as well as sleeve valve openings:

$$H_s = h_f + h_l \qquad \textbf{(Equation 3-9)}$$

Where:

H_s	=	Total system head (m)
h_f	=	Frictional head loss (m)
h_L	=	Secondary head loss (m)

Frictional head losses for the pipes in the intake tower to the downstream sleeve valves were calculated using the Darcy-Weisbach friction loss equation:

$$hf = \frac{\lambda L V^2}{2gD}$$ **(Equation 3-10)**

Where:

$$V = \frac{Q}{A}$$ **(Equation 3-11)**

And:

h_f	=	Frictional head loss (m)
λ	=	Pipe friction factor
L	=	Length of the pipe (m)
V	=	Average velocity in the pipe (m/s)
g	=	Gravitation constant (m/s^2)
D	=	Diameter of the pipe (m)
Q	=	Flow (m^3/s)
A	=	Cross-sectional area (m^2)

The Karman and Prandtl equation for rough pipes was used to calculate the pipe friction factor (λ) rather than the Colebrook-White transition formula, since flow was the unknown parameter:

$$\frac{1}{\sqrt{\lambda}} = -2\log\left(\frac{3.7D}{K_s}\right)$$ **(Equation 3-12)**

Where:

λ	=	Pipe friction factor
K_s	=	Absolute roughness of pipe (m)

The K_s value for steel pipes for this investigation was estimated as 0.5 mm.

The secondary losses were calculated using the following formula:

$$h_L = K\frac{V^2}{2g}$$ **(Equation 3-13)**

Where:

h_L = Secondary head loss (m)

K = Loss coefficient dependant on the fitting type

Fittings and valves contributing to secondary losses include:

- Trashracks and fine screens;
- Entrance loss;
- Bellmouth entrance;
- Butterfly valve;
- 90° bend or T-piece connection to the collector pipe;
- 90° bend of collector pipe;
- Branch piece to smaller sleeve valves;
- Contraction to the sleeve valve;
- Opening of sleeve valve; and
- Exit loss.

The considered pipe size combinations and associated draw down durations are given in **Table 3.39**, assuming there is no inflow into the reservoir and both intake systems are operational to allow for water to be drawn through both of the intake systems simultaneously. The intake levels from where water is to be drawn through depend on the water depth needed to allow for sufficient submergence.

Table 3.39: Considered pipe and valve sizes for the Outlet Works

Pipe Diameter (m)	Sleeve valve diameter (m)	Sleeve valve opening (%)	Time to half depth (days)	Time to LDL (days)
1.8	0.9	100	80	99
1.8	1	75	76	94
2	1	100	65	80
2	1.2	55	65	79
2.1	1	100	65	79
2.2	1.2	80	51	63

The selected combination is 2 m diameter pipes reducing to 1 m diameter sleeve valves which are 100% open during drawdown of the dam. The duration of 65 days to draw the dam down from FSL to half depth is considered acceptable.

The pipe sizing and drawdown calculations are included in **Annexure 3 I** as **Table 3.I.1** and **Figure 3.I.1**, and the draw down curve is shown in **Figure 3.23**. The pipe sizing and drawdown calculations indicate that the outlet works will be able to release a flow of 41.6 m³/s at FSL.

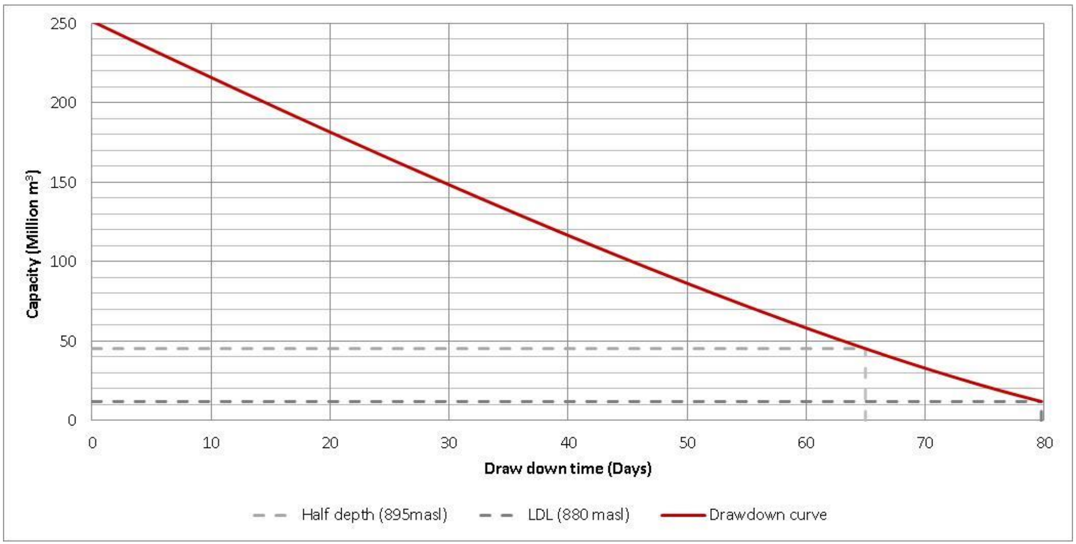

Figure 3.23: Emergency draw down curve

b) EWR releases

The converted daily flows provided a required or target EWR for the river as shown in **Table 3.37**. The possible releases from the spillway and outlet works of Smithfield Dam were modelled with the incorporation of the dam storage capacity. It is shown in **Table 3.40** that with the outlet works capacity of 41.6 m³/s, the target EWR will be closely matched. The deviation from the target EWR is deemed insignificant and the outlet works pipe design is accepted as adequate.

Table 3.40: Target versus dam release availability

Exceedance Probability	EWR Target (m^3/s)	EWR Supply (m^3/s) With Outlet / Release limits	
		Total Supply	Dam Release
100.00%	0	0	0
50.00%	3.6	3.6	2.1
20.00%	9.1	9.1	4.4
10.00%	15.9	15.9	7.5
5.00%	25.6	25.6	12.3
2.00%	43	40	22.3
1.00%	59.9	53.6	30.5
0.50%	83.7	81.1	38.4
0.10%	143.8	137.8	41
0.05%	152.6	148.7	41.2
0.00%	235.2	225.7	41.6

3.13.6 Valves

a) *Intake level selector valves*

The four intake level selector valves are 2 m diameter butterfly valves, which are only used to select the level at which water is to be drawn off. They will thus be either fully open or fully closed. The butterfly valves can be operated locally but remote operation is recommended.

b) *Outlet control valves*

The control valves for releases down the river are situated downstream at the end of the outlet conduit.

Sleeve valves with a 1 m diameter are used to release the EWR and to draw down the dam during emergency conditions. The 1 m pipe will branch into a 300 mm pipe with a 300 mm sleeve valve before the 1 m sleeve valve to release smaller EWRs when required. To contain the spray, hoods are provided with diameters of twice the size of the sleeve valves.

3.14 INTAKE STRUCTURE TO UMKHOMAZI – UMLAZA TUNNEL

3.14.1 General arrangement

The purpose of the Smithfield Dam tunnel intake structure is to house the hydro-mechanical equipment required to operate and control the releases from Smithfield Dam through the uMkhomazi – uMlaza tunnel and raw water pipeline to the Baynesfield WTW and ultimately the Umlaas Road Reservoir.

The intake structure must also provide for the releases associated with the implementation of Phase 2, the construction of the upstream Impendle Dam, and the second tunnel.

The circular tunnel intake structure is positioned within the Smithfield Dam reservoir as indicated in **Figure 3.24**.

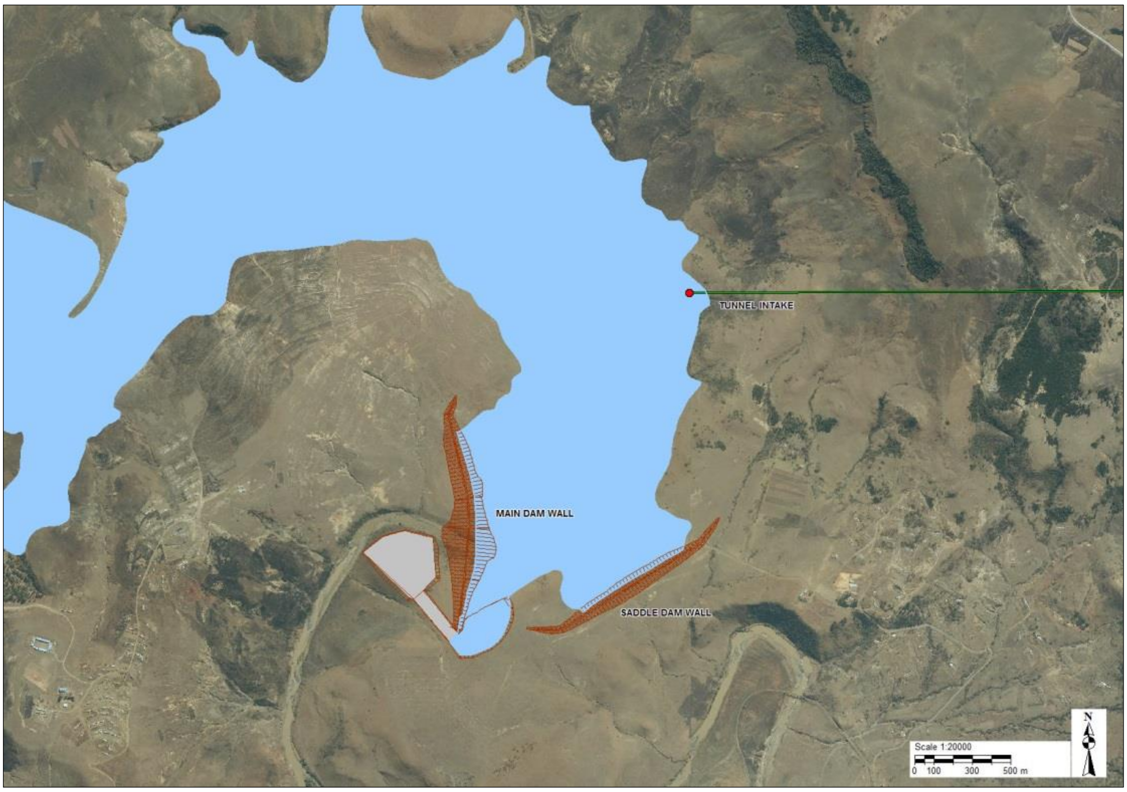

Figure 3.24: Position of the intake structure

The structure connects to the uMkhomazi – uMlaza tunnel with a series of pipes as shown in the general layout (**Figure 3.J.1** in **Annexure 3 J**).

The circular shaped intake structure consists of three intake systems each comprising of multi-level bellmouthed pipes with butterfly valves for selecting the level at which water is to be drawn off. Control valves are situated downstream of each intake system conduit before connecting to the tunnel collector manifold for controlling the releases to the tunnel(s). Service valves (butterfly valves) are positioned in the tunnel collector manifold on both sides of each intake system connection to allow for maintenance and inspection as well as to close off the intake system(s) not in use.

The three-intake system is proposed to accommodate the increased transfer capacity for when Impendle Dam and the second tunnel are implemented as well as to allow for flexible maintenance operation and emergency situations.

Precast concrete trash racks and stainless steel fine screens will be incorporated to prevent floating debris from blocking or damaging the downstream infrastructure. Emergency gates are required for the closure of the bellmouth entrances for maintenance of downstream located valves.

The intake structure also consists of a superstructure which houses the overhead crane required for the operation of the fine screens, butterfly valves and emergency gates. Access to the intake structure will be from the natural embankment via an access bridge.

The intake pipes will connect to the tunnels with bellmouth outlets encased in concrete. Ventilation shafts just downstream of the pipe bellmouth outlets into the tunnels will provide air at the connection to ensure design discharge into the tunnel.

3.14.2 Required transfer capacity

The intake structure is designed to meet the following:

- Phase 1 maximum transfer capacity of 8.65 m^3/s to be conveyed through the uMkhomazi – uMlaza Tunnel.
- Phase 2 maximum transfer capacity of 14.86 m^3/s for the case when the upstream Impendle Dam and second transfer tunnel are implemented. The additional maximum transfer capacity of 6.21 m^3/s is to be conveyed through the second tunnel.

The Phase 1 design transfer capacity is associated with a Smithfield Dam with 31% MAR storage volume and a 1.25 supply peak factor. The proposed maximum design transfer capacity of Phase 2 is based on this Smithfield Dam combined with a 1.5 MAR capacity Impendle Dam and a 1.25 supply peak factor.

Refer to the *Water Resources Yield Assessment Report (P WMA 11/U10/00/3312/2/3)* for more information in this regard.

3.14.3 Minimum operating level

The minimum operating level, to ensure transfer at the designed values, is set at 887.2 masl. The submergence of the intake was taken to be at least 2.5 m to prevent air-entraining vortexes from forming. Thus the crown of the bottom intake was situated below level 884.7 masl.

3.14.4 Sediment deposition at entrance of tunnel

In the Smithfield reservoir the bed level of the approach channel to the intakes is at 881 masl. This level is at the storage volume of the reservoir associated with the expected 50 year sediment volume at a confidence level of 80% (13.91 million m³ – refer to the *Sediment Yield Report (Water Resources Yield Assessment Report: Supporting Document 1)*) retained in the reservoir from the Smithfield Dam embankment with a horizontal depositing pattern. The sediment deposition in the Smithfield Dam basin is discussed in the *Sediment Deposition and Impact Report (Water Resources Yield Assessment Report: Supporting Document 2)*.

This assumption is not practical as most silt will be deposited in the upper reaches of the reservoir where the water velocity of flowing water decreases and where streamflow power allows. As a result, a sedimentation deposition study considering the impact of sedimentation around the reservoir intake to the tunnel over a period of more than 100 years should be carried out. Furthermore, it is preferable to carry out this study during this feasibility stage than the detail design stages of the project in order to prevent changes to the vertical alignment of the tunnel during the design stages, which may have an effect on the yield of the system when the minimum operating level has to be raised. If this study is not carried out in any phase of the implementation of the project it may result in the tunnel entrance becoming blocked or sediment being drawn through the tunnel and supply pipelines in the future. Therefore, the depositing of silt is a study on its own using specific principles, and must be done before the commencement of the final design.

3.14.5 Layout requirements

a) *Three intake system*

The intake structure consists of three intake systems of which system 1 and 2 will be operational with the implementation of Phase 1 of the uMkhomazi Project to feed the uMkhomazi – uMlaza tunnel. This dual system is required for maintenance purposes.

System 3 will accommodate the additional release requirement when Phase 2 of the uMkhomazi Project is implemented in approximately 2044. This system will be constructed to a level where this system can be blocked off and linked to the other two systems when it becomes necessary. This includes the bellmouth intakes on the various levels, the section of pipe leading from the intake structure to the second tunnel and the bellmouth outlet at the pipe-tunnel connection. The ventilation shaft leading from the intake structure will be constructed during Phase 1. The remainder of pipes and valves of system 3 will be installed during the construction of Phase 2.

b) *Multi-level intakes*

From the *Water Quality and Limnological Report (P WMA 11/U10/00/33/3312/3/1)*, six abstraction levels were proposed to ensure the best possible water quality is abstracted. These abstraction levels are shown in **Table 3.41**. Each of the three intake systems consist of intakes at these abstraction levels. Generally inlets are staggered between intake systems but due to the costs related to the number of intake systems and practicality in this regard, it is proposed that the intakes not be staggered and rather be positioned beneath each other. A bottom inlet is proposed with its invert level at 881.5 masl for water to be abstracted down to the MOL.

Table 3.41: Centreline level of intakes

Abstraction level	Meters above sea level (masl)	Intervals (m)
L1	924.0	6.0
L2	918.0	6.0
L3	912.0	6.0
L4	906.0	6.0
L5	898.0	8.0
L6	890.0	8.0
Bottom inlet invert level	881.5	8.5

Note: FSL at 930 masl

Tunnel inlet invert level at 881 masl

c) *Intake bays*

Three intake bays are provided, one for each of the three intake systems. The intake bays will extend outward from the main circular structure. The intake bays have been sized to ensure sufficient approach flow area at the entrance of the intake bay and hence an acceptable flow velocity is maintained through the trashracks and fine screens.

d) *Trashracks and fine screens*

Trashracks and fine screens are designed to prevent floating trash and debris, mostly occurring near the water surface, to be drawn into the intakes and damage equipment.

The upstream end of the three intake bays are protected by fixed coarse precast concrete trashracks followed by removable downstream stainless steel fine screens.

The fine screen panels are lowered into guides embedded in concrete piers. These removable fine screen panels are each fitted with a tray at the upstream bottom to collect trash or debris when the screens are hoisted for cleaning purposes.

A grappling beam for handling the screens will be provided with storage in a rack on the deck.

e) *Emergency gates*

Emergency gates are required to close off the bellmouth intakes to allow for inspection and maintenance thereof. Due to the layout of the intake structure, each of the intake systems should comprise of an emergency gate as interchanging an emergency gate between the three intake systems is not permitted.

f) *Drywell*

A circular shape was proposed for the intake structure as it provides more seismic resistance than a square or rectangular structure. Furthermore, compressive stresses rather than tensile stresses are induced on the structure, resulting in less concrete reinforcing being required.

The dry well will house the following:

- Intake level selector valves (butterfly valves);
- Control valves (knife gate valves) of each intake system;
- Service valves (butterfly valves) for closing off intake systems and regulating the flow direction;
- Elevator shaft as well as stair cases to provide access to the valves on the various intake levels; and
- Inlet of the ventilation shafts at the pipe-tunnel connections.

g) *Super structure and overhead crane*

A superstructure is located above the intake structure and houses an overhead crane to handle the fine screens, emergency gates and the various valves during installation, removal or refurbishment.

Access to the superstructure is permitted via an access bridge from the tunnel inlet portal access road. The bridge is supported with columns spaced every 12 m. The superstructure can be reached by a 10 ton truck for the transportation of the valves, emergency gates and fine screens.

Sufficient overhead cranes for the handling of the valves are provided.

h) *Ventilation shafts*

A 3.5 m diameter steel pipe ventilation shaft is provided for each of the tunnels downstream of the pipe bellmouth outlets. The ventilation shafts are encased in concrete with the inlets situated within the intake structure. The purpose of the ventilation shafts is to provide a facility for air entrainment to ensure undisturbed water flow conditions. More detail pertaining to the ventilation shafts and air entrainment into the tunnels is discussed in **Section 4.2.10**.

3.14.6 Sizing of pipework and valves

The pipes and associated valves were sized based on the following considerations:

- The maximum flow velocity permitted through the various valves; and
- The required head at the Baynesfield WTW must be at a minimum of 872 masl.

a) *Maximum flow velocity*

The maximum flow velocity in the pipes is limited to 7 m/s to prevent excessive noise, vibration and wear, especially of the butterfly valves.

The design flow velocities associated with the Phase 1 and Phase 2 transfer capacities in three considered pipe sizes are given in **Table 3.42**.

Table 3.42: Associated velocities in various diameter pipes

Pipe diameter	Phase 1 flow velocity (8.65 m^3/s release)	Phase 2 flow velocity (6.21 m^3/s release)
1.4	5.62	4.03
1.6	4.30	3.09
1.8	3.40	2.44

b) *WTW head requirement*

The head requirement at the Baynesfield WTW must be at a minimum of 872 masl to provide water under gravitation to the Umlaas Road Pipeline.

The MOL of Smithfield Dam is 887.2 masl. Hence, if water is abstracted from one of the bottom intakes, the friction and secondary losses incurred from the upstream side of the intake structure to the WTW should be limited to 15.2 m.

The critical flow path where most losses will be incurred is considered to be from the bottom intake of System 2. This path has the longest flow length, accumulating the most friction loss, and passes the most pipe fittings which contribute to secondary or minor losses.

Friction losses for the pipes in the intake structure to the pipe-tunnel connection were calculated using the Darcy-Weisbach friction loss equation:

$$h_f = \frac{\lambda L V^2}{2gD}$$ **(Equation 3-14)**

Where:

h_f	=	Frictional head loss (m)
λ	=	Pipe friction factor
L	=	Length of the pipe (m)
V	=	Average velocity in the pipe (m/s)
g	=	Gravitation constant (m/s²)
D	=	Diameter of the pipe (m)

The Colebrook-White equation was used to calculate the pipe friction factor (λ):

$$1/\sqrt{\lambda} = -2\log\left(\frac{ks}{3{,}7D} + \frac{2{,}51}{RE\sqrt{\lambda}}\right)$$ **(Equation 3-15)**

Where:

$$RE = \frac{DV}{v}$$ **(Equation 3-16)**

And:

λ	=	Pipe friction factor
k_s	=	Absolute roughness of pipe (m)
Re	=	Reynolds number
D	=	Pipe diameter (m)
V	=	Velocity in pipe (m/s)
v	=	Kinematic viscosity (1.13 x 10^{-6} m²/s)

The k_s value for steel pipes for this investigation was estimated as 0.5 mm.

The secondary losses were calculated using the following formula:

$$h_L = K\frac{V^2}{2g}$$ **(Equation 3-17)**

Where:

h_L	=	Secondary head loss, m
K	=	Loss coefficient dependant on the fitting type
V	=	Average velocity in the pipe, m/s

Pipe fittings contributing to secondary losses from the upstream side of the intake structure to the pipe bellmouth outlet into the tunnel include:

- Trashrack and fine screens;
- Bellmouth entrances;
- Butterfly valves;
- 90° bend to the vertical collector pipe;
- T-pieces passed in the vertical collector pipe;
- 90° bend in collector pipe;
- Control valves;
- Y-piece (connection of collector pipe to tunnel collector manifold);
- T-pieces passed in tunnel collector manifold;
- 125° bend of tunnel collector manifold; and
- Bellmouth exit into tunnel.

Losses incurred from the start of the uMkhomazi – uMlaza Tunnel to the Baynesfield WTW are discussed in **Section 4.3**.

A spreadsheet containing head loss calculations and relevant loss coefficient factors for the conveyance of releases from the upstream side of the intake structure to the Baynesfield WTW is included in **Annexure 3 J** as **Table 3.J.1**.

c) Conclusion and recommendations

From the aforementioned considerations and calculations it is recommended that the pipe sizes from the bellmouth intake to the collector pipe and tunnel collector manifold connection be 1.6 m in diameter for all intake levels of system 1 and system 2. The sizes of the pipes of the same section of system 3 can be reduced to 1.4 m in diameter.

The tunnel collector manifold is recommended to be 1.8 m in diameter leading into the tunnels.

The total friction and secondary losses accumulated from the upstream side of the intake structure to the Baynesfield WTW are 15.2 m.

Refer to **Annexure 3 J** for layouts of the pipe sizes and the determination thereof.

3.14.7 Valves

a) Intake level selector butterfly valves

Each of the intakes is equipped with a 1.6 m diameter butterfly valve just downstream of the bellmouth inlet, except for the intakes of system 3 which will have 1.4 m diameter butterfly valves. The butterfly valves are only used to select the level at which water is to be drawn off and will thus be either fully open or fully closed.

b) Intake system control valves

The control valves for the releases into the tunnels are situated upstream of each intake system connection to the tunnel collector manifold.

The 1.6 m valves will be partially opened or closed according to the intake system and level from which water is abstracted to ensure the correct releases through the required tunnels.

c) Service valves

Service valves (butterfly valves) of 1.8 m diameter within the tunnel collector manifold will be used to control the flow path of the releases as well as to close off the intake systems not in use. It must be noted that the service valves downstream of the connection of system 2 to the tunnel collection manifold leading to the second tunnel will only be installed during the construction of Phase 2. Until then a blank flange will be placed on the tunnel collector manifold between the system 2 and system 3 connections to only permit releases into the first uMkhomazi – uMlaza tunnel.

The butterfly valves will only be used to close off an intake system or pipe section and will thus be either fully open or fully closed.

3.15 STABILITY OF SLOPES AROUND RIM OF RESERVOIR

3.15.1 Background

Reservoirs surrounded by steep unstable slopes are subject to landslides that can displace material into the reservoir causing volumetric displacement of water and setting up surges and waves in the water body which can lead to overtopping of the dam. Volumetric displacement by material can be dealt with as an incoming volume and subsequently leading to a rise in water level and capacity reduction. Calculation of the slip volume possibly threatening the dam can be made from a geological analysis of the surrounds of the basin. Three types of slips occur according to *Vischer* (1986), namely (i) falls such as rock masses off a cliff with low volume and high energy intensity, (ii) slides such as slip-circle type slides also known as debris-flow and (iii) more gradual flows which are associated with long time intervals.

Not listed in the above types is the mechanism that caused the disastrous slide into the Vajont reservoir in Italy in 1963 when a large part of the mountain (260 000 000 m³) on the left side slid along a curved bedding plane into the relatively small reservoir at a speed of about 30 m/s and created a wave of over 250 m high that swept 50 000 000 m³ of water that destroyed a town on the right bank and also spills over the crest of the dam. More than 2 500 people were killed on the right bank and downstream of the dam.

Huber and Hager (Huber & Hager, 1997) developed a generalised approach for estimating impulse waves under general conditions. Their earlier work is used for the calculation of wave heights in the SANCOLD Guidelines on Freeboard in Dams (South African National Committee on Large Dams, 1990). Also, Hager is the main author of a guideline for the calculation of landslide generated impulse waves in reservoirs, published by the Laboratory of Hydraulics, Hydrology and Glaciology of the Swiss Federal Institute of Technology (VAW, 2009).

Input parameters to be obtained from a geological and topographical study are the location, volume, width and density (compactness) of potentially unstable material and the inclination of the sliding plane. From the guidelines by VAW these parameters, together with information on water depth and positions of the critical impact areas (e.g. dam wall), can be used to predict wave heights at critical locations and wave run-up against slopes (e.g. dam walls).

3.15.2 Geology and rock types

The areas around the main dam and saddle dam and in parts of the dam basin are underlain by near-horizontally bedded rocks of the Volksrust Formation, Ecca Group, while the other parts of the dam basin are underlain by near-horizontally bedded rocks of the Estcourt and Adelaide Formations of the Beaufort Group. These formations belong to the Karoo Supergroup. Dolerite sills had intruded these sedimentary strata mostly concordantly, while a few sub-vertical dolerite dykes are present.

The Volksrust Formation comprises dark grey shale, interbedded with subordinate sandstone. Dark grey to black carbonaceous shale, siltstone and sandstone occur in the Estcourt Formation while the Adelaide Formation comprises siltstone, sandstone and subordinate shale.

3.15.3 Investigation of the dam basin

The dam basin is located in an area where the uMkhomazi River had incised a deep valley into the surrounding landscape. Its tortuous course is structurally controlled by main joint sets in the sedimentary rocks and resistance provided by dolerite dykes and dolerite sills. The insides of bends usually have gentle slopes while outside bends have steeper slopes due to undercutting by the river.

The dam basin is about 12 km long as measured along the river, and the surface area of the reservoir at FSL is 9.53 km^2. The top 1 m layer of the dam basin thus represents a volume of 9 530 000 m^3. It also means that if 9.5 million m^3 of material gradually slides into a full dam, the water level will rise by about 1 m. If the slide takes place quickly, a higher impulse wave might occur and its effect will depend on the position of the slide with respect to the dam wall.

The investigation took place in February and March 2014 and was split into the following phases:

a) Desk study

The desk study involved an inspection of geological maps, topographical maps and satellite imagery (Google Earth). 17 potential unstable slope areas were identified on the basis of slope angle (steeper than 25 degrees) and the position of the steep section along the slope.

The positions of the 17 potential slide areas are shown on **Figure A7.2** included in **Annexure A** of the *Smithfield Dam Construction Materials and Geotechnical Investigation*.

b) *Field investigation*

During the field inspection of the identified slopes, a GPS was used to determine the positions and elevations of points above the 930 masl contour. Although not very accurate (barometric heights), these points were used to determine the gradient of these slopes above FSL.

From the geological map and field visits, the rock types forming the slopes were identified. Where possible, the orientations and continuity of major joint planes that intersect the rock faces were inspected. Unfortunately in some cases the slopes were covered by scree or very dense vegetation and no rock outcrops were visible.

c) *Analysis*

Of the 17 identified slopes, it was found that only 4 were steeper than 25 degrees and also in direct line of sight from the main or saddle dams. According to *Huber and Hager* (1997), wave action from slides that are out of sight due to topographic features will have little impact on structures. One slope (Slope S13) that is out of sight of the dam walls was identified as a potential slide that might result in large volumetric displacement and overtopping of the dam.

The method proposed by *VAW* (2009) was used to calculate the wave height and wave run-up at the centre of the main and saddle embankments as a result of complete rapid failure of each potential slide area. The results are given in **Table 3.43**.

Table 3.43: Probability of failure and effect on dam walls

Slope area no.	Type of sliding material	Volume of potential sliding mass (m^3)	Probability of failure	Wave height main dam (m)	Run-up main dam (m)	Wave height saddle (m)	Run-up saddle dam (m)
2	Gravel*	4 000	moderate	0.00	0.00	0.16	0.18
	Shale	40 000	extr. low	0.00	0.00	0.67	0.92
6	Talus	80 000	moderate	1.15	1.41	0.09	0.09
	Shale	200 000	extr. low	2.12	2.84	0.15	0.18
6a	Gravel*	6 000	moderate	0.21	0.21	0.02	0.02
	Shale	500 000	extr. low	3.13	4.40	0.08	0.09
7	Talus	36 000	moderate	0.00	0.00	0.58	0.77
	Shale	60 000	extr. low	0.00	0.00	0.88	1.17
13*	Talus	120 000	moderate	0.01	0.01	0.01	0.01
	Shale	880 000	extr. low	0.10	0.10	0.10	0.10

**Note: Rise in water level due to displacement – no impulse wave.*

3.15.4 Conclusions and recommendations

The geotechnical report concludes that there is a moderate (1:50 year) probability of a talus/gravel failure from slopes (Areas 6 and 6a that are located about 1.5 km from the dam walls) that will result in a run-up wave of up to about 1.4 m against the main dam wall. There is also an extremely low (1:10 000 year) probability of a large rock slide from the same slope area that will result in a run-up of about 4.4 m against the main dam wall.

Despite the small probability of these events occurring, design procedures should be included to mitigate them should they occur. Thus, the freeboard of the embankment and saddle dam should be such that it would be able to prevent overtopping of the NOC in the event of such failures. Failures of the other identified slopes will have much smaller effects due to smaller potential slide volumes, longer distances from the dam walls and topographic barriers between the slide areas and the dam walls.

3.16 Upgrading of Transmission Lines

The current 88 kVA transmission line from Bulwer to Elandskop crossing the proposed Smithfield Reservoir should be changed to accommodate the 700 m span over the to-be-impounded reservoir. This line is due to be upgraded in approximately 10 years' time to a 132 kVA line.

3.16.1 Design

The conceptual design of the proposed upgrade will probably consist of new towers at the sides of the reservoir designed for 220 kVA or 400 kVA. The design should, however, include obtaining towers with sufficient attachment height that will result in the towers having sufficient clearance for either 88 kVA or 132 kVA. The existing towers will have to be dismantled.

3.16.2 Procedures to be followed

The customer executive for ESKOM is to be contacted to request a transmission line deviation project (Mr Navie Reddy currently holds this position (2014)). All related ESKOM projects for this water project should be known in order to determine how they all fit together. This will help to identify load requirements for construction supplies and permanent supplies at each point. The lead time should not be more than two or three years from the start of the project. The regional ESKOM office in Pietermaritzburg is the entity with decision-making power to undertake this work.

3.16.3 Cost estimate

A rough estimate of the preliminary cost to undertake this design and construction work is R 5 million. This accounts for the demolishing of the towers in the reservoir. The upfront payment to ESKOM is 100% of the project cost and the lead time for the payment is dependent on environmental studies and negotiations.

3.17 RECOMMENDATIONS

The following recommendations are applicable to Smithfield Dam:

- The excavation for the founding level of the main embankment will yield a large volume of material, which might be suitable as impervious and semi-pervious earthfill for the saddle embankment. Laboratory testing of this material will have to be conducted to confirm the suitability.
- A grout curtain at the saddle embankment is recommended to a level at least 20 m below the floor of Quarry I, due to the development of this quarry just upstream of the saddle embankment. Although grout penetration might be very small, the drilling, water test and grout records from a grouting operation is very important and can be considered the final stage of a geotechnical

investigation when sub-surface information is obtained at close intervals below the footprint of the dam.

- Additional geotechnical investigations are required in the tender design phase to determine foundation conditions for the position of the main spillway as well as the erosion potential at the fuse plug spillway. Subsequently a total cost optimisation of the dam freeboard and spillway width should be carried out.
- The anchors and drainage system for the spillway chute should be designed in the detail design phase.
- The chute will end with a ski jump discharging into a plunge pool. The ski jump for Impofu Dam was adopted for the feasibility design and will have to be optimised during the tender design phase.

3.18 Principal Smithfield Dam Data

The principal data for Smithfield Dam is summarised in **Table 3.44**. **Figure 3.25** and **Figure** 3.26 are artistic impressions of Smithfield Dam during the operational phase.

Table 3.44: Smithfield Dam principal data

Parameter	Description	
General		
Name	Smithfield Dam	
Purpose	Bulk water supply	
Estimated date of completion	2022	
River	uMkhomazi River	
Nearest town	Bulwer	
District	KwaZulu-Natal	
Location	29°46'33.36" S; 29°56'26.62" E	
Classification: Category	III	
Size class	Large	
Hazard potential	High	
Non-overspill crest level	RL 936 masl	
Full supply level (FSL)	RL 930 m	
Gross storage capacity at FSL	251 million m³	
Water surface area at FSL	9.53 km²	
	Main wall	**Saddle wall**
Wall height above river level (Maximum height)	81 m (855 masl to 936 masl)	26 m (910 masl to 936 masl)
Type of dam wall	Earth core rockfill	Zoned earthfill
Crest length	1 200 m	1 090 m

Parameter	Description	
Spillway type	Side channel	Fuse plug
Spillway form	Ogee	Broad crested
Spillway length	150 m	100 m
Freeboard	6 m	2 m
Hydrology and floods		
Catchment area	2 058 km²	
Safety evaluation flood	5 650 m³/s	
Regional maximum flood	4 540 m³/s	
$Q_{1:100}$	2 389 m³/s	
$Q_{1:200}$	2 620 m³/s	
Outlet works		
Dam Outlet	Dual pipe system of ND 1.8 m, 6 intakes, Butterfly and gate valves	
Tunnel Inlet	Tri pipe system of ND 2 m, 6 Intakes, Butterfly and gate valves	
Description of dam wall foundations	The site comprises of shales (mudrocks) with sub-ordinate sandstones and intrusions of dolerite. Three near-horizontal dolerite sills have intruded mainly concordantly into the sedimentary strata and are responsible for the narrow river valley at the dam site and the presence of good quality rock for concrete aggregate and rockfill.	

Figure 3.25: View 1 of Smithfield Dam's artistic impression

Figure 3.26: View 2 of Smithfield Dam's artistic impression

3.19 COST ESTIMATE

A detailed cost estimate of all construction activities of Smithfield Dam, comprising quantities and rates, has been completed, and is contained in **Annexure 3 K**. **Table 3.45** shows a summary of this cost estimate. Assumptions made in determining all cost estimates are described in **Section 14**. The total scheme cost estimate with all components added together is given in **Section 14.4**.

Table 3.45: Summary of cost estimate of all activities for Smithfield Dam

Description	Cost (R million, excl. VAT)
River diversion works	178.5
Development of quarries and borrow areas	9.9
Smithfield Dam main embankment (zoned earth core rockfill dam)	813.5
Smithfield Dam saddle embankment (zoned earthfill dam)	252.1
Main embankment side channel spillway	189.7
Saddle embankment fuse plug spillway	66.0
Outlet works, intake structure	146.4
Tunnel intake structure	288.4
Miscellaneous	73.2
TOTAL	**2 017.8**

3.20 REFERENCES

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4 uMkhomazi – uMlaza Tunnel

The conveyance system from Smithfield Dam to the Baynesfield WTW consists of a pressure tunnel and a raw water pipeline – as discussed in *Supporting Document 1: Engineering Feasibility Design,* called *Optimisation of Conveyance System Report*. A plan layout of this system is shown in **Figure 4.1**.

This chapter describes the tunnel regarding the following:

- The geological conditions in the tunnel area;
- Tunnel and shaft design philosophy;
- Hydraulic design;
- Selected tunnel layout;
- Portal design; and
- Summary of the components of the tunnel and cost estimate.

4.1 Geological conditions in the tunnel area

4.1.1 Introduction

This section discusses some of the key findings in the *Geotechnical Report: Supporting Document 5* for the foundations and construction materials present at the locations of the intake tower and inlet portal, the free-flow tunnel, the access/ventilation adits, ventilation shafts and the outlet portal. For specific detail, the abovementioned report should be consulted.

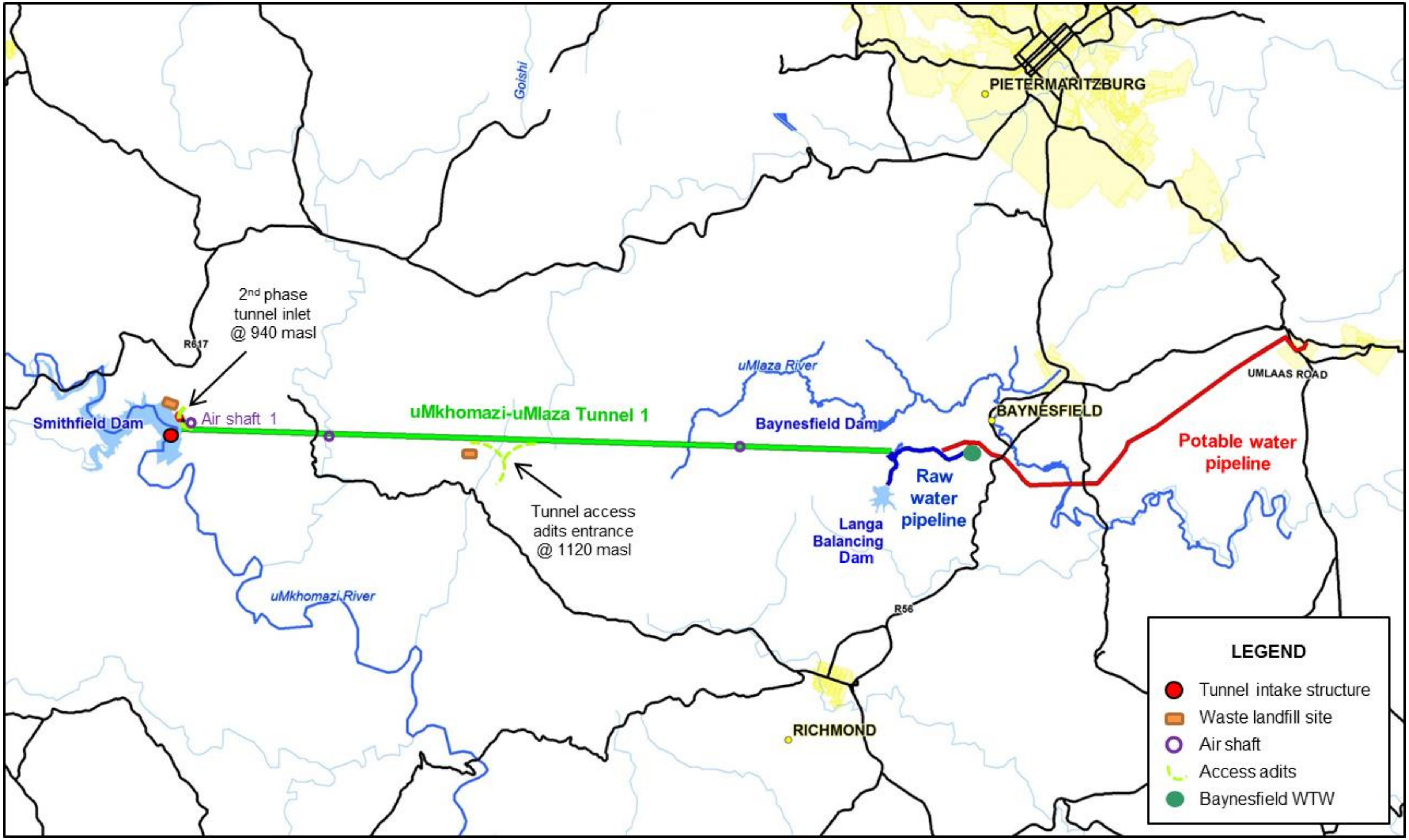


Figure 4.1: Proposed layout of the conveyance system

4.1.2 Intake tower and inlet portal geotechnical evaluation

The weathered rock in the area around the intake tower and tunnel inlet portal was found at a depth of approximately 5 m below ground surface with less weathered and fractured rock appearing at 15 to 20 m depth below ground surface. However, in some areas slightly fractured to moderately weathered shale rock is encountered at approximately 6 m below the ground surface. The shale rock is soft to hard and highly fractured.

Test pit profiles indicated that weathered shale rock is overlain by clayey silt and colluvium. The dolerite rock and Volksrust Formation contact zone were not encountered in any of the boreholes or trial pits. The water table in the area is close to the surface with seepage evident in places. All excavations will need to be adequately supported or flattened and dewatering measures will need to be put in place to facilitate both earth and concrete works.

a) Soil material properties

The results from laboratory testing on soil samples taken at the intake tower and inlet portal indicate that the soils classify as SC (clayey sands, poorly graded sand clay mixtures) or CH (inorganic clays of high plasticity). The SC material is considered to be a suitable material for engineered fills and for use as selected or sub-grade in pavement layers. However, the CH material is not considered suitable for use in engineered fills and is a poor material to use in pavements, even as sub-grade. The CH material may be utilised in the construction of the dam wall.

b) Weatherability and protection of cut slope faces

Shale rock will be exposed on the excavated rock faces of the inlet portal. As the soft rock shale will tend to weather relatively quickly, a sprayed concrete (shotcrete) layer should be applied as a weather guard to these portions of the excavated face. It is unlikely that any form of weather guard would be required on the excavated faces in hard to extremely hard rock shale.

Rock material from the tunnel inlet area and access tunnels will be excavated by means of drill and blast techniques (DBT). This will result in blast rock with variable particle sizes. The more weathered rock material may be utilised in engineered fill or in pavement layerworks. The unweathered shale may be utilised in engineered fill or rock fill, but cognisance should be taken of the fact that this material, especially the carbonaceous shale, is liable to weather in time, especially if it is subject to wetting and drying cycles. If dolerite is encountered, the unweathered dolerite rock may be utilised in either a rock fill, as rip-rap, or crushed and utilised as aggregate.

c) Rock joint sets

A joint survey was carried out on exposed rock faces in the vicinity of the inlet portal. The results from this survey are summarised in the geotechnical report. These results will be utilised to determine the level of rock support required to stabilise the excavated rock faces in the portal area and indicate that the joint sets are generally sub-vertical and sub-horizontal.

4.1.3 Tunnel

Boreholes positioned along the tunnel line indicated that dolerite sills and/or dykes were not prevalent. The boreholes were primarily drilled in fractured hard to extremely hard rock shale, with a number of boreholes encountering some hard to extremely hard dolerite.

The high probability of encountering extremely hard rock dolerite sills and dykes within the strong sedimentary host rock makes it likely that fairly uniform face conditions would be encountered, which would be beneficial for the Tunnel Boring Machine’s (TBM’s) cutter life and excavation times. Furthermore, the anticipated shallow dip of the sedimentary strata will likely ensure that dolerite sills persist within the excavation profile over considerable distances. Joints in the shale rock are mostly smooth, whilst joints in the dolerite rock are mostly rough.

The presence of near-vertical faults and dykes, together with an anticipated high water table and variable cover (generally between 20 and 680 metres), indicate that significant groundwater inflows should be expected. Intergranular and fractured aquifers may be expected in the Karoo supergroup rocks.

a) Potential construction materials

Rock material from the tunnel will be excavated predominantly by means of TBMs. The TBM sections will produce a more uniform aggregate than sections excavated by means of DBT. However, the cutting disks on the TBM tend to produce a flakey aggregate. The unweathered dolerite may be utilised in a rock fill, but cognisance will have to be taken of the flakey nature of the aggregate.

Alternatively, the unweathered dolerite may be crushed and utilised as aggregate. The unweathered shale may be utilised as general fill, but again care will have to be taken with regards to the flakey nature of the aggregate and the fact that this material, especially the carbonaceous shale, is likely to weather in time, especially if it is subject to wetting and drying cycles. During excavation of the tunnels, one would either have a mixed face condition where both shale and dolerite are present, or where only shale or dolerite is present. It is anticipated that extensive lengths will be excavated in either shale or dolerite – so it would be reasonably easy to stockpile these two types of material separately.

4.1.4 Central access/ventilation adits and ventilation shafts

Boreholes were drilled at proposed ventilation shaft positions, to determine the depth to bedrock, the rock mass quality and stratigraphy. All boreholes indicated the presence of hard to extremely hard rock shale and very hard to extremely hard rock dolerite at tunnel level. Hard bedrock was generally encountered at depths between 10 and 20 m below ground surface.

In general, the rock at the above locations tended to be highly fractured and jointed within the first 40 m to 50 m. The carbonaceous shale rock tended to be more jointed with the joints being partly open to very tight. Joints in the shale rock were mostly smooth while joints in the dolerite rock were mostly rough. Dykes are expected and water bearing fracture zones trending northwest and north may be encountered below the water table. Without any pre-grouting, significant water inflow may be expected in the event that a water bearing fracture is struck.

Access adits at the mid-point, driven from the valley bottom at a gradient of 1V:10H, would need to be between 1 500 m and 2 000 m long.

a) Rock material properties

The boreholes employed to investigate the access and ventilation shafts are the same as those utilised to investigate the condition of the rock mass and stratigraphy along the tunnel route. A review of the borehole logs indicated that there are zones of relatively soft moderately to highly fractured shale rock above the competent rock found along the proposed tunnel line.

Isolated zones of very soft shale rock are more pronounced at shallow depths (i.e. closer to the surface) but pockets were encountered in one of the boreholes approximately 200 m below the surface.

b) *Excavation of the ventilation shafts*

The ventilation shafts may be excavated either by means of conventional shaft sinking techniques, or by raise-boring.

Conventional shaft sinking techniques would entail drilling and blasting the rock mass and advancing the shaft from ground surface. The spoil material would be hoisted out of the shaft as the shaft advances to depth. Alternatively, the shaft may be raise-bored. The advantage of this method is that the shaft can be excavated (or bored) relatively quickly. However, the disadvantages are, firstly, that raise-boring can only commence once the TBM has reached the intersection with the ventilation shaft; secondly, the raise-bored material has to be removed past the TBM; and thirdly, the TBM is unproductive while raise-boring is ongoing.

c) *Weatherability and protection of cut slope faces*

Shale and/or dolerite rock will be exposed on the excavated rock faces of the intermediate access portal and in the ventilation shafts. As the soft rock shale and carbonaceous shale will tend to weather relatively quickly, it is recommended that a sprayed concrete (shotcrete) layer be applied as a weather guard to these portions of the excavated face. It is unlikely that any form of weather guard would be required on the excavated faces in hard to extremely hard rock shale or dolerite.

4.1.5 Outlet portal

The borehole log and test pit profiles indicate that the area around the outlet portal is overlain by colluvium (up to 2.6 m thick) and residual shale rock down to a depth of approximately 9.0 m. The residual shale rock generally consists of either gravelly sandy silt or clayey silt. The residual shale rock is in turn underlain by a 2.3 m thick zone of highly fractured moderately to completely weathered laminated soft grey shale rock, which in turn is underlain by highly weathered fractured laminated soft dark-grey shale rock.

The bedrock consists of highly weathered and fractured shale rock at depths of 20 to 25 m below surface. However, geophysical results from west of the outlet portal suggest that fresh shale rock is encountered at 35 m below surface. This is overlain by 5 m of highly weathered and fractured shale bedrock, which in turn is overlain by residual shale rock and colluvium.

The outlet portal is located on Baynesfield Estate land. Some areas close to the outlet portal have been designated as wetlands by the Department of Environmental Affairs. Therefore any earthworks or concrete works will have to face strict environmental guidelines and measures will have to be put in place to prevent the setting off of any environmental triggers. All excavations will need to be adequately supported or flattened and dewatering measures will need to be put in place to facilitate both earth and concrete works

a) Soil material parameters

The results from laboratory testing on soil samples taken at the outlet portal indicate that the soils classify as MH (inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts) or OH (organic clays of medium to high placticity, organic silts). This material is not considered suitable for use in engineered fills and is a poor material to use in pavements, even as sub-grade.

b) Rock joint sets

A joint survey was carried out on exposed rock faces in the vicinity of the outlet portal. The results from this survey are summarised in the geotechnical report. These results will be utilised to determine the level of rock support required to stabilise the excavated rock faces in the portal area and indicate that the joint sets are generally sub-vertical and sub-horizontal.

c) Weatherability and protection of cut slope faces

Shale rock will be exposed on the excavated rock faces of the outlet portal. As the soft rock shale will tend to weather relatively quickly, it is recommended that a sprayed concrete (shotcrete) layer be applied as a weather guard to these portions of the excavated face. It is unlikely that any form of weather guard would be required on the excavated faces in hard to extremely hard rock shale.

4.2 TUNNEL AND SHAFT DESIGN PHILOSOPHY

4.2.1 General considerations

Design and construction of tunnels and shafts in rock requires processes and procedures that are in many ways different from other design and construction projects, because the principal construction material is the rock mass itself rather than an engineered material. Uncertainties persist in the properties of the rock materials and in the way the rock mass and groundwater will behave. These uncertainties must be overcome by flexible design and safeguards during construction. More than for any other type of structure, the design of tunnels must involve the selection of an anticipated method of construction.

The selection of a TBM construction arrangement in comparison with a DBT construction methodology is based on a lower construction cost and a significantly shorter time required for excavation of the tunnel. More information is provided in **Annexure 4 A**.

4.2.2 Geology and hydrogeological considerations

The site geology provides the scene for any underground structure. The mechanical properties of the rock mass determine how the geologic materials deform and fail under the forces introduced by the excavation. The hydrogeological conditions establish the quantity and pressure of the water to be controlled.

The materials and geotechnical investigation carried out for the feasibility design indicated that the tunnel will traverse sub-horizontally bedded shale rock of the Karoo Sequence which has been extensively intruded by dolerite. The 1:250 000 geological survey maps indicate a number of faults, generally striking NW to SE at roughly 45° to the tunnel alignment. The shale rock at tunnel level is unweathered and very hard to extremely hard, whilst the dolerite is extremely hard and unweathered. Both rock types are described as slightly to highly fractured.

The hydrogeological conditions along the tunnel route vary considerably. During investigations the groundwater was generally located at depths between 20 and 30 m; however, artesian conditions were encountered in two of the exploratory rotary core boreholes – one borehole was dry and two had relatively shallow ground water tables. The high groundwater flows were encountered where the rock mass was highly fractured.

Although no specific testing was carried out to determine the presence of gas, this cannot be precluded as some carbonaceous shale rock was encountered in some boreholes.

4.2.3 Construction of the tunnels

Methods and sequences of tunnel excavation affect the loads and displacements exerted by the rock mass that have to be resisted by the rock support. Although it is good practice to leave many details of construction for the contractor to decide, it is often necessary for the designer to specify methods of construction when these affect the quality, cost, programming or safety of the work. The basic components of underground construction include, inter alia, the following:

- Excavation, by means of blasting or by mechanical means;
- Ground support;
- Survey;
- Site and portal preparation;
- Ventilation and lighting of the underground works; and
- Drainage and water control.

The central access adits of the tunnel, required for access to the main tunnel during excavation and for maintenance under operation, span 5 m to provide sufficient space for machines to access the tunnel and transporting components of the TBMs. These adits will not be lined; however, they will be covered with a layer of shotcrete where necessary. A section through the access adits is shown in **Figure 4.2** as well as in a drawing in **Annexure 4 B** as **Figure 4.B.1**.

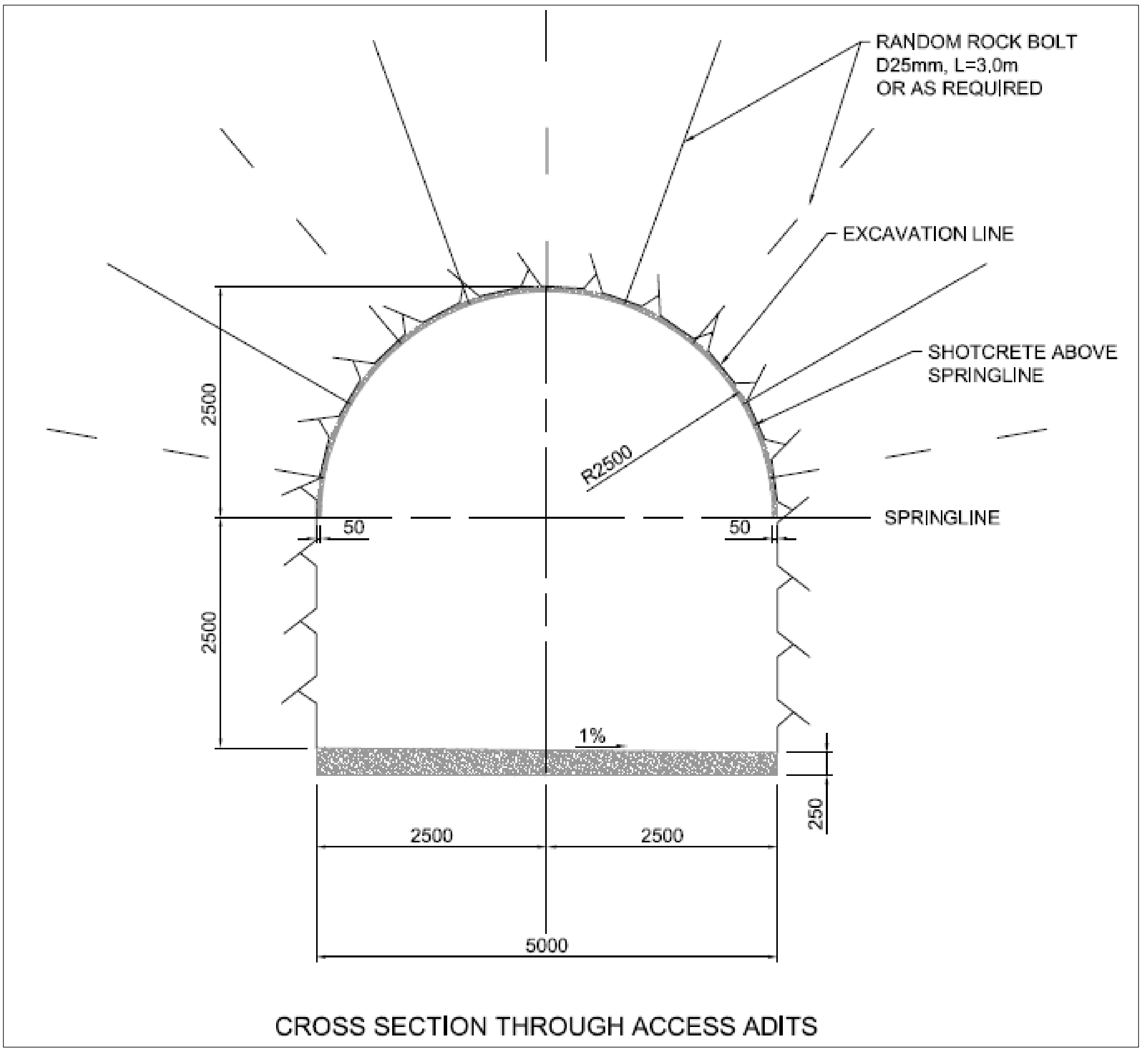


Figure 4.2: Section through an access adit

Reasons for the large size of the access adits:

- Sufficient room for transporting components of the TBM;
- Sufficient space provisions for vehicles moving in and out during operation, with some enlarged areas to facilitate passing of construction vehicles; and
- Sufficient space for mucking conveyors, ventilation ducts, lighting and services.

A portion of the main tunnel in the vicinity of the central access adit will be enlarged to a 7.5 m span to facilitate the assembly and dismantling of the TBM. This enlargement will be required over a length of 40 to 50 m. This area will also be utilised for the storage of conveyor belting, pre-cast lining segments, ventilation ducts and services.

Due to the length of the water transfer tunnel and the fairly uniform strength of the rock mass, this tunnel will be excavated by two double shield TBMs. These machines consist of two main components, a front shield with the cutterhead, main bearing and drive, and a rear shield with gripping unit, auxiliary thrust cylinders and tailskin. The main thrust cylinders connect the two parts of the shield.

In stable rock, the machine is braced radially against the tunnel with the gripper shoes in the rear shield while the front shield is advanced independently of the gripper shield using the main thrust cylinders. Simultaneously while tunnelling, the precast liner segments are installed in the tailskin section. When the advancing stroke is completed, the gripper shoes are released and the rear shield is pushed towards the front shield utilising the auxiliary thrust cylinders. Regripping lasts only a few minutes, which means that tunnelling is almost continuous.

Double shield TBMs can also operate in fault zones or areas of weak rock. In these sections radial bracing is not possible and the front and rear shields are retracted to form a rigid unit. The thrust force to advance the TBM is applied using the auxiliary thrust cylinders which push against the last segment ring installed. As a result, tunnelling and ring building can no longer take place simultaneously. However, this method provides for higher tunnelling safety in difficult sections of the alignment.

It is envisaged that no primary support will be installed as the final lining will be installed fairly close to the excavation face. The final tunnel support will consist of precast lining segments that will be assembled to form watertight rings. The annulus between the assembled rings and the excavated rock face will be backfilled with cementitious grout. The assembled rings will have a reasonably low Manning’s number which will assist with water flow and velocity criteria.

Two TBMs will be utilised on this project, the first advancing from the outlet portal to a central access adit and the second from the central access adit towards the inlet portal. Both drives will be undertaken up-grade to ensure drainage of the tunnels.

Advantages of using a TBM include the following:

- Higher advance rates;
- Continuous operations;
- Less damage to the rock mass;
- Fewer support requirements;
- Uniform muck characteristics; and
- Greater work safety.

Disadvantages of a TBM are the fixed circular tunnel geometry, limited flexibility in response to extremes in geological conditions, longer mobilisation time and higher capital cost.

Due to the proposed layout of the tunnel and to accommodate access for the tunnel boring machines, it is proposed to excavate some of the tunnels by means of DBT, i.e. the access tunnels (adits) and the central portion of the water transfer tunnel. The DBT is suited to the excavation of these tunnels as it is versatile in being able to excavate tunnels of varying dimensions and more adept at handling varying ground conditions. The tunnels will be advanced full face and vibrations from blasting are not considered to be problematic due to the rural location of the project.

Primary rock support for the DBT sections of the tunnel will incorporate either tensioned or un-tensioned rock bolts with varying thicknesses of either fibre or mesh reinforced sprayed concrete, i.e. shotcrete. The use of steel arches or lattice girders is not foreseen. The final tunnel support will consist of a cast in-situ concrete lining which will be reinforced where it has to cater for high internal or external water pressures. As mentioned above, the access adits will not be lined; however, they will be covered with a layer of shotcrete where necessary.

4.2.4 Access during construction

The radius of the access tunnels should be at least 250 m with a vertical gradient of 12% to facilitate the transport of TBM equipment.

As it is envisaged that a second tunnel will be excavated around 2044, it is prudent to excavate the inlet stub tunnel during the initial contract to facilitate access to the second main tunnel as this inlet will be inundated in 2044 by the Smithfield reservoir. The inlet from the intake tower and shaft of this stub tunnel should be provided with stoplogs or an equivalent sealing system, as this will be necessary for the excavation of the second tunnel without interfering with the operation of the initial tunnel.

4.2.5 Portal construction

Tunnels usually require a minimum of one or two tunnel diameters of rock cover before tunnelling can safely commence. To start with, an open excavation is made which when finished will provide the necessary cover to begin tunnelling. Rock reinforcement systems are often used to stabilise the rock above the tunnel.

Tunnel excavation from the portal should be done carefully and judiciously with controlled blasting and short rounds being used initially. After the tunnel has been advanced to two to three diameters from the portal face, or as geology dictates, the blasting rounds may be increased progressively to standard length rounds for normal tunnelling.

When constructing portals, the following should be taken into account:

- The rock in the portal area is likely to be more weathered and fractured than that in the tunnel.
- The portal must be designed with proper regard to slope stability conditions, since the portal excavation will unload the toe of the slope, and saturated/drawdown conditions will exist at the inlet portal.
- The portal will be developed at the beginning of mining, as the excavation crew gain experience and go through their learning curve.
- The portal will be a heavily utilised area and a conservative design approach should be taken because of the potential negative effects any instability would have on the tunnelling operations.
- Where TBMs are to be assembled, the portal area should allow for a level area of 40 to 50 m length with a minimum width of 7.5 m.

The design of the portal reinforcement will depend on geological conditions and both rock and soil stability analyses will be required as both types of material are present.

4.2.6 Shaft construction

Most underground works include at least one deep excavation or shaft for temporary access and ventilation or as part of the permanent facility. In this scheme, a minimum of two access shafts will be required, initially for ventilation purposes and in the permanent case for venting during de-watering, air entrainment during streamflow and to act as surge chambers if necessary.

As shafts are excavated from ground surface, they typically go through a variety of ground conditions which include overburden excavation, weathered rock and unweathered rock of various types, with increasing groundwater pressure. Shafts serving permanent functions (personnel access, ventilation or utilities, drop shaft, de-airing, surge chamber, etc.) are sized for their ultimate purpose.

Shallow shafts through overburden are often large and rectangular in shape. Deep shafts servicing tunnel construction are most often circular in shape with a diameter as small as possible. Considering the services required for the tunnel work (hoisting, mucking, utilities, etc.) typical diameters are between 5 and 10 m. If a TBM is used, the shaft is usually sized to accommodate the largest single component of the TBM, usually the main bearing, which is approximately two-thirds of the TBM diameter.

Shaft excavation through soil overburden may be carried out utilising conventional soil excavation methods such as backhoes and dozers, supported by cranes for muck removal. Many options are available for initial ground support, including, inter-alia, the following:

- Soldier piles and lagging in soils where groundwater is not a problem or is controlled by dewatering;
- Ring beams and lagging;
- Cast in-situ concrete lining utilising a tapered sliding shutter;
- Precast segmental shaft lining;
- Steel sheet pile walls, often used in wet ground that is not too hard for driving the sheet piles;
- Diaphragm walls cast in slurry trenches; generally more expensive but used where they can form part of the permanent structure or where settlements and groundwater must be controlled;
- Secant pile walls or soil-mixing walls as substitutes for diaphragm walls, which are generally less expensive where they can be used; and
- In good ground above the water table, soil nailing with sprayed concrete (i.e. shotcrete) is often a viable ground support alternative.

Circular shafts constructed with diaphragm, secant pile or cast in-situ concrete walls usually do not require internal bracing or anchor support, provided circularity and continuity of the wall is well controlled.

Conventional shaft sinking methods are generally utilised for excavation in rock. Blasting techniques can be used to construct a shaft of virtually any depth, size, and shape. A circular shaft is usually preferred, because the circular shape is most favourable for opening stability and lining design. Shallow shaft construction can be serviced with cranes, but deeper shaft construction requires more elaborate equipment. The typical arrangement includes a headgear at the top, suspending a two- or three-story stage with working platforms for drilling and blasting, equipment for mucking, initial ground support installation, and final shaft lining placement. The typical shaft lining is a cast-in-situ concrete lining, placed 10 to 15 meters above the advancing face.

Most shaft construction requires the initial construction of a shaft collar structure that supports overburden and weathered rock near the surface. It also serves as a foundation for the temporary headgear used for construction as well as for permanent installations at the top of the shaft.

4.2.7 Ground improvement

When difficult tunnel or shaft construction conditions are foreseen, ground improvement is often advisable and sometimes necessary. There are, generally, three types of ground improvement that can be feasibly employed for underground works in rock formations:

- Dewatering;
- Grouting; and
- Freezing (which is not seen as an option on this project).

In overburden or weathered material, ground improvement must be considered when shaft sinking involves unstable ground associated with significant groundwater inflow. If sufficiently shallow, the best solution is to extend the shaft collar, consisting of a nominally watertight wall, into the top of the rock. Shallow groundwater can also often be controlled by dewatering.

Deep groundwater cannot be controlled by dewatering. This is usually done by cementitious grouting from the ground surface to full depth before shaft sinking commences, because it is very costly when carried out from within the shaft. The detailed grouting design for deep shafts is often left to a specialist contractor to perform and implement. Grout penetration into rock fractures is limited by the aperture width of the fractures relative to the cement particle size. As a rule, if the rock formation is too tight to grout, it is also usually tight enough that groundwater flow is not a problem.

Shaft grouting usually starts with the drilling of two or three rows of primary grout holes around the shaft perimeter, spaced 2.0 m to 2.5 m apart. Grout injection is performed in the required zones usually from the bottom up, using packers. The effectiveness of the grouting can be verified by drilling secondary grout holes, which if they display little or no grout take, is a sign of the effectiveness of the grouting. Where required, additional grout holes can be drilled and grouted until results are satisfactory. A limit of 1.0 Lugeon (obtained from packer testing in the grout hole) is usually considered satisfactory to ensure adequate water tightness.

Rock tunnels generally do not require ground improvement as frequently as shafts. Where it is known that the tunnel will traverse weak ground with high water pressure, the ground can be grouted ahead of time. It is preferable to grout from the ground surface, if possible, to avoid delaying tunnelling operations. The primary purpose of grouting is to reduce the ground permeability; strengthening of the ground is sometimes an additional benefit.

When grouting cannot be carried out from the ground surface, it can be carried out from the face of the tunnel before the tunnel reaches the region with adverse conditions. Where adverse conditions are anticipated but their location is unknown, probe hole drilling will help determine their location and characteristics. Where required, an arrangement of grout holes is drilled in a fan shape some 20 to 40 m ahead of the face. Quality control is achieved by drilling probe holes and testing the reduction in permeability. Grouting is continued until a satisfactory reduction is achieved.

If it is found that water inflow into the excavated tunnel is too large for convenient placement of the final lining, radial post-grouting can be performed to reduce the inflow. Generally, the grout is first injected some distance from the tunnel, where water flow velocities are likely to be smaller than at closer distances. Where it is necessary to perform radial grouting after the completion of the tunnel lining, the finished lining helps to confine the grout, but the lining must be designed to resist the grout pressure.

4.2.8 Drainage and control of groundwater

Prior to construction, estimates of the expected sources of groundwater and the expected inflow rates and volumes must be identified in order for the contractor to provide adequate facilities for handling inflow volumes. Water occurring in the tunnel during construction must be disposed of because it is a nuisance to workers and machinery. When encountered, water should be channelled to minimise its effect on the remaining works. Where possible, all tunnels should be excavated up-slope to ensure that they are free draining. Where necessary, water will have to be pumped out of tunnels that are excavated down-slope.

The water transfer tunnel on this project has a slope of 0.027% and when completed will be free draining. The channels, however, must be cleared from silt during construction, only the lower section where the one TBM enters from the outlet portal will be free draining. The upper section where the TBM enters from an access tunnel will be free draining down to the access tunnel, from where it will have to be pumped to the surface. Due to the length and pumping height of the access tunnel, this may entail the excavation of a series of side adits with sumps to facilitate continuous pumping.

If excavation of a tunnel is carried out with free drainage away from the excavation head and no pumping of the water is required, the cost of excavation is approximately 10% less.

4.2.9 Construction of permanent tunnel lining

When the initial rock support components do not fulfil the long-term functional requirements for the tunnel, a final lining is installed. In the DBT sections, the final lining will typically be constructed of cast-in-situ concrete, reinforced or unreinforced, or a steel lining surrounded by concrete or grout. For the TBM section, the initial ground support consisting of precast segments will also serve as the final lining. This scheme is fully lined as both the shale and dolerite are generally slightly to highly fractured. It can be reconsidered during construction that the lining of the tunnel be left out for portions of the tunnel that are not significantly fractured.

4.2.10 Ventilation of tunnels and shafts

Shaft and tunnel construction generally occurs in closed, dead-end spaces, and forced ventilation is essential to the safety of the works. Contractors are responsible for the safety of the work, including temporary installations such as ventilation equipment and their operation and are obliged to follow the law as enforced by the Occupational Health and Safety Act (OSHA). The purpose of underground ventilation during construction serves at least the following purposes:

- Supply of adequate quality air for workers;
- Dilution or removal of construction-generated fumes;
- Cooling of air – heat sources include equipment and high temperature of rock/groundwater; and
- Smoke exhaust in the event of underground fire-dust control.

In the permanent structure, ventilation provisions may be required for at least the following purposes:

- To bleed off air at high points in the alignment;

- To purge air entrained in the water, resulting, for example, from aeration in a drop shaft; and
- To provide ventilation for personnel during inspection of empty tunnels.

These ventilation requirements often result in the use of permanent ventilation shafts with appropriate covers and valves.

A drawing showing the cross-sectional view through a ventilation shaft has been included in **Annexure 4 B** as **Figure 4.B.1**, alongside a section of an access adit.

4.2.11 Spoil management

Disposal of material removed from tunnels and shafts is often the source of considerable discussion during the environmental planning phase. The spoil on this project is to be placed in the provided waste landfill site areas as discussed in **Section 10**.

Equipment and construction may generate waste waters that require statutory permits to discharge into surface waters. In any event, all construction water will need to be treated before being discharged into a natural watercourse.

In addition to the water generated by construction, it is expected that existing ground water will be encountered during the tunnelling process. This water is required to be removed from the construction area and discharged back into a natural resource, assuming it is not contaminated. Permits will be required to allow the return of the water to a natural system and will be covered during the environmental studies on the tunnel. If the encountered groundwater is contaminated, then provision for mitigation like a water treatment package plant should be made to ensure the contaminated water does not enter the existing water resources in the area.

4.2.12 Practical considerations for the planning of tunnel and shaft projects

For many tunnels, size, alignment, and grade are firmly determined by functional requirements, such as for gravity water transfer tunnels.

For rapid and economical tunnelling of relatively long tunnels, a bore diameter of approximately 4.5 metres (3.5 metres for horseshoe shape) or larger should be adopted. This tunnel size permits the installation of a California switch to accommodate a 1.07 metres gage rail, which allows passing of reasonably sized train cars.

Shafts excavated by blasting should be at least 3.5 metres in size.

Rubber tired equipment can conveniently negotiate a 10% grade, but up to 25% is possible. The usual maximum speed is about 40 km/h.

For conveyor belts, a grade of 17% is a good maximum, though 20% can be accommodated with spoil that does not roll down the belt easily. Depending on belt width, the maximum particle size is 0.3 to 0.4 m. Most belts run straight, but some modern belts can negotiate large-radius curves.

Usually, shafts shallower than 30 m employ cranes for hoisting; a headgear is used for deeper shafts. Vertical conveyors are used for muck removal through shafts for depths greater than 120 m.

4.2.13 Economic tunnel drive lengths

A study conducted on the Mohale Tunnel of the Lesotho Highlands Project has shown that 15 km is the most economical length of drive achievable by a 4.5 m outside diameter TBM. Thus, it is envisaged that at least two TBMs would be utilized on this project. Aspects such as access and ventilation can become problematic with longer drives.

As the tunnel(s) will operate under pressure, it is assumed that they will be fully concrete lined along the entire length. Waterproof membrane and steel liners have not been considered necessary at this stage. The assumptions should be refined at detail design stage once more data is available.

4.2.14 Progress rates

The TBM excavation and concrete lining are anticipated to be at an advance rate in the order of 130 m per week, per heading. This would equate to an excavation duration of approximately 123 weeks (or 2.4 years) for two TBMs.

The lead time required to get a TBM on site would need to be added to this. Lead time varies from 9 to 12 months from time of order for a new machine, to perhaps 3 to 9 months for a refurbished machine. Once the machinery is on site, 3 to 6 weeks will be required for assembly.

The start of mining operations rarely occurs with the full back-up system in place. Hence, decreased advance rates, of the order of 50% of full production, should be expected for the first 4 to 8 weeks of mining. As the full back-up system is installed and the TBM crew learns the ropes of system operation versus ground conditions, full production may be expected within 2 months.

Experience indicates that tunnel depth has little impact on advance rates in civil projects, provided that the contractor has installed adequate mucking capacity for non-delay operation. Therefore, tunnel depth should be chosen primarily by location of good rock. Portal access, as opposed to shafts, will facilitate mucking and material supply, but it is more important that the staging area for either shaft or portal be adequate for contractor staging (such as precast yard if segmental lining is to be utilised). Confined surface space can have a severe impact on project schedule and costs.

4.2.15 Effect of tunnel on groundwater regime

The tunnel would first be constructed and later be operated after construction.

The construction of the tunnel would entail:

- Drilling or blasting of rock, and opening of possible aquifers especially at weathered rock or rock contacts where water is expected to be encountered;
- Immediate sealing of areas where excessive groundwater is expected by grouting of rock or providing mass concrete plugs where necessary;
- Concrete lining of the full length of the tunnel. This must be reconsidered during detail design; and
- Conveyance of excessive seepage water during construction and discharging this water into the stream (a permit is required for this).

During operation, the tunnel water will be conveyed under pressure. No groundwater is expected to be added to the water conveyed from Smithfield Dam. It may, however, be necessary to pipe groundwater behind the liners from high fractured and leaking zones through a separate pipe system for discharge into the stream.

As the geotechnical investigations (boreholes) carried out during the feasibility design stage are still limited to 4 to 6 holes to the tunnel alignment area of a 32.5 km long stretch, it is not possible to project the quantity of expected seepage into the tunnel area. Normally this rate is very small and compared to the total mass of rock and groundwater on top of a 32.5 km long and approximately 400 m deep mountain area the expected seepage of say 5 l/s is small and insignificant and is not expected to impact significantly on the water head. It is also not expected to impact the quantity of boreholes which may only be 60 to 100 m deep at some locations in this rock mass.

Furthermore, the shales are horizontally layered which could cause water to flow vertically only at fractured zones. The dolerites, expected to be up to 40% of the tunnel line, are normally water tight except at contact zones with shales and/or vertical joints or fractures.

It is therefore concluded that:

- Minimum and insignificant effects on the groundwater and yields of boreholes can be expected.
- The highest risk of encountering insignificant effects is during construction in times when the boring machine has completed drilling. The grouting is done from the second “train truck” behind the bore and a liner is put in place from the “third truck” – a small time span.
- High fluoride water was encountered in one borehole at 60 m depth during investigations. This location is more than 400 m away from the tunnel. The quality of water encountered in the tunnel will only be known during construction. If required, a treatment facility should be constructed. This aspect must be added to the tunnel tender for construction.

4.3 HYDRAULIC DESIGN

4.3.1 Layout and assumptions

The hydraulic design of the water conveyance tunnel and pipeline is based on the available pressure head from Smithfield Dam, as well as the maximum design flow requirement of 8.65 m^3/s.

A longitudinal layout of the structures with the associated energy line is shown in **Figure 4.3**.

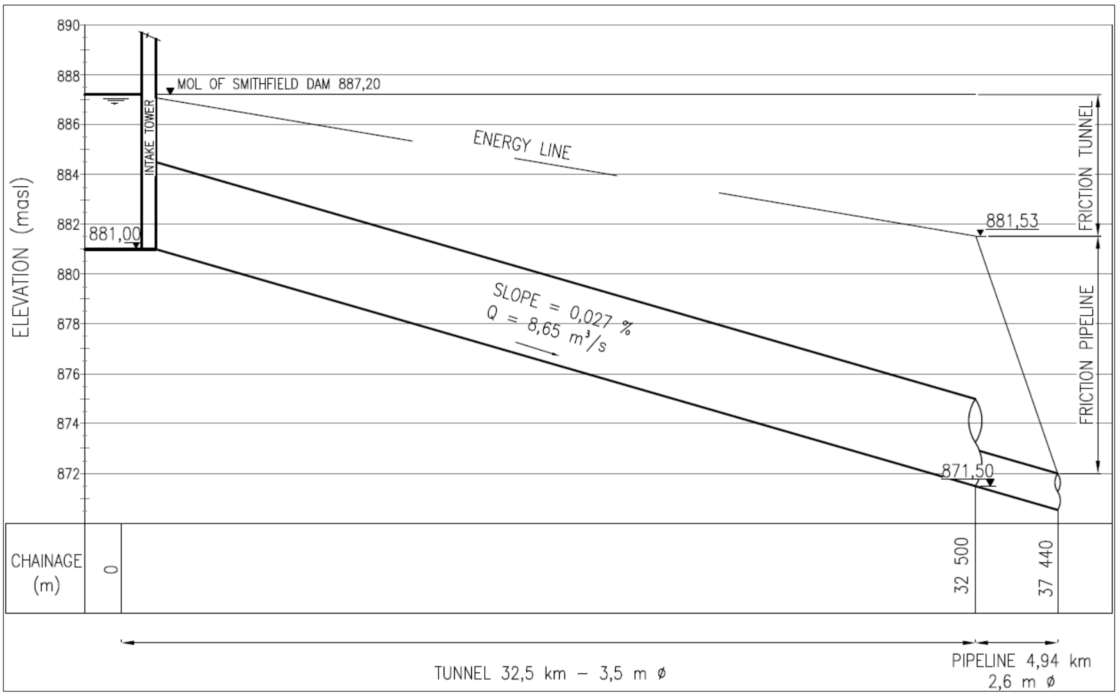


Figure 4.3: Schematic layout of conveyance system showing the energy line

This schematic layout is based on the following:

- The MOL of Smithfield Dam (887.2 masl) is the minimum water level at the upstream side of the intake tower to the tunnel.
- The intake centre level of the lowest pipe at the tunnel intake structure is at 881.5 masl.
- A lined tunnel with an inside diameter of 3.5 m, cross sectional area of 9.616 m² and length of 32.5 km as indicated in **Figure 4.3** and **Figure 4.6**.
- The Tunnel – Langa Dam – Baynesfield Pipeline is a 2.6 m inside diameter pipe from the tunnel end to Baynesfield WTW and has a length of 5.12 km.
- A stilling basin at the end of the pipeline at the Baynesfield WTW with a tailrace water level of 872 masl. This is required to provide water under gravitation to the Umlaas Road Pipeline (Infrastructure from WTW to Umlaas Road is described under *Module 3: Technical Feasibility Study: Potable Water*).
- The peak design flow is 8.65 m³/s.
- Provision has been made at the inlet portal for the construction of two tunnels – one of which will be constructed during the second phase of the project.
- Ventilation shafts have been provided for in the design to accommodate air flow in the tunnel. One ventilation shaft is provided at the entrance of the tunnel and two others have been included on either side of the central access adits (which are approximately in the middle of the tunnel).

A general layout of the raw water conveyance pipeline from the tunnel outlet to Baynesfield WTW has been included in **Annexure 4 C** as **Figure 4.C.1**.

The components of the proposed conveyance system – inclusive of both the tunnel and pipeline, excluding the intake tower discussed earlier – having an effect on hydraulic friction consist of the following:

- A 1.8 m bellmouth exit at the pipe outlet into the conveyance tunnel;
- The 3.5 m diameter tunnel for a length of 32 500 m;
- A reducer from the 3.5 m diameter tunnel to the 2.6 m diameter steel pipe;
- Two 90° bends;
- A 2.6 m diameter butterfly valve;
- A 2.6 m diameter steel pipe from the tunnel outlet to the Baynesfield WTW with a length of 4.94 km; and
- A USBR Type IV stilling basin at the outlet of the steel pipe.

4.3.2 Hydraulic assessment

The friction formula for the tunnel and the pipe is the Darcy-Weisbach friction loss equation (**Equation 4.1**):

$$h_f = \frac{\lambda L V^2}{2gD} \qquad \textbf{(Equation 4-1)}$$

Where:

h_f	=	Frictional head loss (m)
λ	=	Pipe friction factor (dimensionless)
L	=	Length of the pipe (m)
V	=	Average velocity in the pipe (m/s)
g	=	Gravitation constant (m/s²)
D	=	Diameter of the pipe (m)

The formula for determining the pipe friction factor (λ) is the Barr pipe friction factor equation (**Equation 4.2**):

$$1/\sqrt{\lambda} = -2\log\left(\frac{k_s}{3.7D} + \frac{2.51}{Re\sqrt{\lambda}}\right) \qquad \textbf{(Equation 4-2)}$$

Where:

$$RE = \frac{DV}{v}$$ **(Equation 4-3)**

And:

k_s = Absolute roughness of the pipe (m)

Re = Reynolds number (dimensionless)

v = Kinematic viscosity (1.13×10^{-6} m²/s)

The k_s value for a tunnel was estimated for this investigation as 1.5 mm and for a steel pipe as 0.5 mm. The minor losses in the tunnel were estimated as $0.5\frac{V^2}{2g}$ and for the steel pipe as $0.8\frac{V^2}{2g}$.

4.3.3 Summary of hydraulic assessment results

The design in a spreadsheet format is included in **Table 3.J.1** in **Annexure 3 J**. In accordance with this calculation all head losses are met for a discharge of 8.65 m³/s and a 3.5 m internal diameter tunnel.

4.3.4 Air entrainment considerations

It is necessary to provide air entrainment in the tunnel so as to avoid any unwanted pressures and the effects they may cause. In order to implement the provision of air entrainment into the tunnel system, three ventilation shafts were designed. **Figure 4.4** indicates Ventilation Shaft 1, which is a 5 m diameter pipeline connected to the tunnel above the inlet pipe; the entire drawing from which this figure is taken has been included in **Annexure 3 J** as **Figure 3.J.2**.

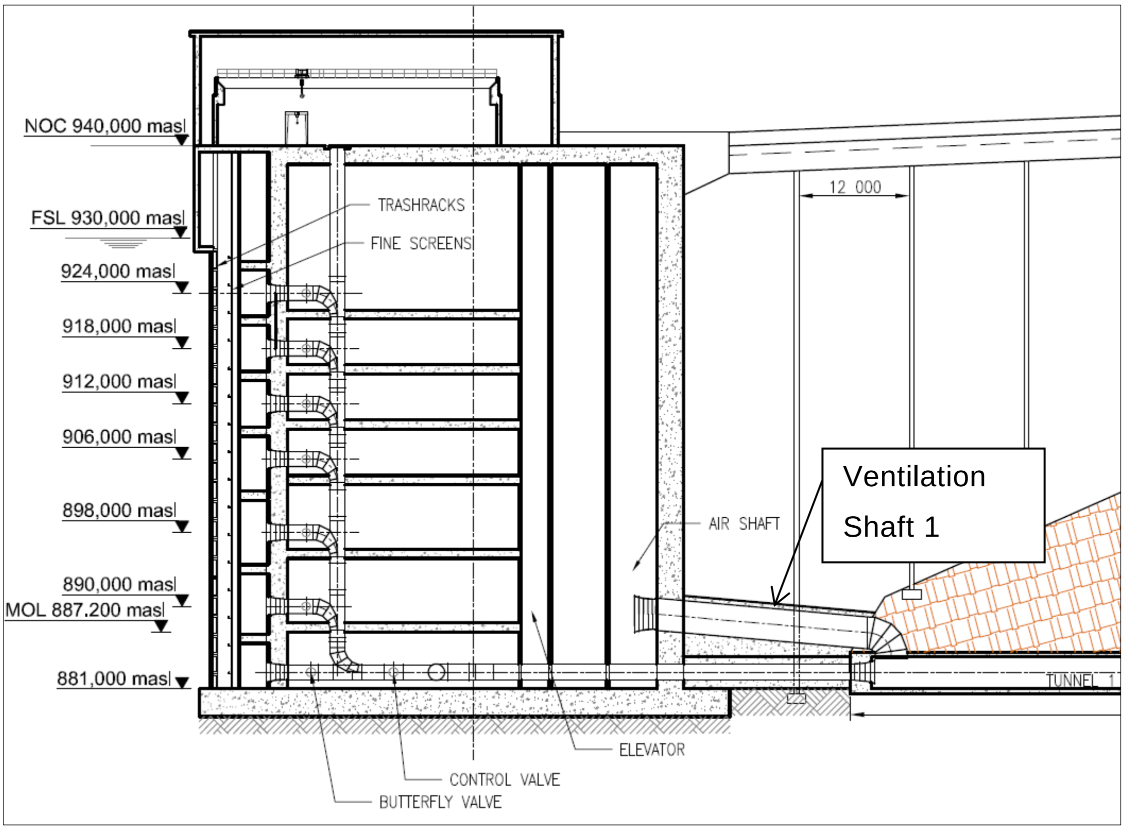


Figure 4.4: Section through tunnel inlet showing provision of air entrainment measures

Ventilation Shafts 2 and 3 are located as indicated in **Figure 4.5**. These vents are 5 m diameter vents which will be excavated to the tunnel level through DBT excavation and, where necessary, lined by shotcrete. As previously mentioned, a cross sectional view of the vents has been included in **Annexure 4 B** as **Figure 4.B.1**.

In addition to the three ventilation shafts, two access adits will be excavated by DBT close to the middle of the tunnel length. The exact location of the adits can be seen in **Figure 4.5**. The access adits start together and then split into adits 1 and 2 after approximately 700 m. The adits are a horseshoe shape, where the upper sections are 5 m in diameter with a square base with a width of 5 m. **Figure 4.2** shows a section through an access adit. These adits will provide air entrainment into the tunnel in addition to access.

4.4 ALIGNMENT OF SELECTED TUNNEL

4.4.1 Horizontal alignment

In transferring water from the proposed Smithfield Dam in the uMkhomazi River Valley to the uMlaza River Valley the uMkhomazi – uMlaza Tunnel was laid out on the shortest straight route. Initially a route to the upper reaches of Baynesfield Dam was considered, but discarded mainly due to the expensive geological problems that would have been encountered when laying a pipeline in high ground on the right side of the dam, also in saturated conditions. This is described in the Optimization Report in more detail.

A route, similar in cost, ending at the upper reaches of the new Mbangweni Dam was selected. For this route, areas for the inlet and the outlet portals were selected on flatter mountain slopes where access would be possible and areas for assembling the TBM could be made.

However, large areas at the portals would have to be excavated in soft materials. The cost for excavating and stockpiling material and adding pipes of these areas are, however, less than to bore and line rock in these areas with TBMs. The pipeline lengths are also shorter than the initial Baynesfield route. These aspects and comparisons are described in *Supporting Document 3: Optimisation of Scheme Configuration (P WMA 11/U10/00/3312/3/1/3)*.

4.4.2 Vertical alignment of tunnel

In this section the vertical alignment of the tunnel is optimised further, focussing on local geological and potential high groundwater inflow conditions, construction methods, practical conditions and drainage aspects. The following have been addressed:

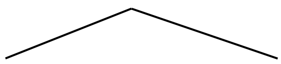
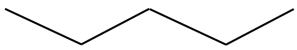
- Engineering geology;
- Expected tunnel conditions;
- Excavation method;
- Size of tunnel;
- Advance rates;
- Drainage during construction and operation of scheme;
- Access to the tunnel;
- Different alignment options;

- Position and level of the intake for EWR, drainage and tunnel releases during the operation of the scheme; and
- Costs.

4.4.3 Vertical alignment options

The identified alignment options with drainage considerations and comments are shown in **Table 4.1**, with initial cost estimate comparisons of the options shown in **Table 4.2**.

Table 4.1: Description of tunnel alignment options

Option	Configuration	Direction of excavation and drainage requirements during construction	Drainage requirements during operation
1	Slopes downwards from centre towards ends  Intake foundation level: 862 masl Outlet level: 875 masl	TBM accesses from the ends. No drainage requirements for free draining conditions	Pumping or drainage for inspection required for up-slope half. Ventilation shaft required in centre.
2	One downward slope  Intake foundation level: 883.87 masl Outlet level: 871.5 masl	Upper half to be driven from centre in an up-slope direction. Pumping of drainage water from centre.	Free draining: ventilation shaft required at entrance.
3	Slopes to meet the 0.1% criteria  Intake foundation level: 883.87 masl Outlet level: 871.5 masl	Drainage towards low points and pumping from these points.	Pumping from lower points.

In the initial cost estimate (shown in **Table 4.2**) the following has relevance:

- The unit cost of the tunnel was adjusted in accordance with the complexity of driving below water conditions.
- Vertical alignments of the tunnels meet the hydraulic and construction drainage requirements.
- The additional cost for the intake tower for Option 1 relates to a deeper foundation compared to the other options.
- The tunnel drainage pipe is necessary to drain the up-slope part of the tunnel for Option 1 through the reservoir of the dam and through the outlet of the dam. This is, however, not favoured from a maintenance point of view. Draining of the tunnel can also be done by pumps from the bottom of the

intake tower. This option is also not favoured due to pumps which may not be available for pumping when needed.

- Option 2 is the option with the lowest total cost.

Table 4.2: Initial cost estimate comparison of vertical alignment options

Activities	Quantity	Unit Cost	Amount (R million, excl. VAT)		
			Option 1	Option 2	Option 3
Intake Tower	Sum	-	88.7	76.5	76.5
U/S part of tunnel (m)	14 651	83 000	1 216.0	-	-
U/S part of tunnel (m)	14 651	86 624	-	1 269.1	1 269.1
D/S part of tunnel (m)	17 927	83 000	1 487.9	1 487.9	-
D/S part of tunnel (m)	17 927	86 624	-	-	1 552.9
Additional excavation cost at intake tower (m^3)	308 000	100	30.8	-	-
Tunnel drainage pipe to dam outlet	3 500	20 000	70	-	-
Additional ventilation shaft	SUM	-	-	2	-
Total	**-**	**-**	**2 893.5**	**2 835.6**	**2 898.5**

Option 2 therefore suits the construction and operational requirements best.

4.4.4 Selected layout of the tunnel

The selected horizontal and vertical layout of the tunnel is shown in **Figure 4.5**; the proposed tunnel section with chainages has also been included in **Annexure 4 D** as **Figure 4.D.1**. This layout is based on the following:

- One vertical slope;
- Excavations at both portals of the tunnel;
- A DBT access adit at the central part of the tunnel from chainage 14 750 m to 16 250 m;
- Two TBM drives, both up-slope in a western direction;
- Two 5 m diameter ventilation shafts with shotcrete lining and a 5 m access adit in the centre;
- One 5 m diameter ventilation shaft with concrete lining near the entrance to the tunnel for phase 1 (tunnel 1);
- Three tunnel waste disposal landfill sites (including one at Langa Dam);
- An access adit at the entrance to facilitate access to the tunnel for phase 2 (tunnel 2); and
- Construction of the first 100 m of the tunnel 2 to ensure access during full Smithfield Dam conditions.

The water will be abstracted from Smithfield Dam into the tunnel intake structure through 14 bellmouth intake pipes. However, 21 bellmouth intakes are to be provided for the system, with only 14 being used until the implementation of the second phase of the project.

From the tunnel intake structure the water is transferred through a 1.8 m pipe and a bellmouth and into the 3.5 m diameter tunnel. The inlet portal will be excavated using DBT, as will the excavation required for Ventilation Shaft 1, which enters the top of the tunnel close to the inlet (as indicated by **Figure 4.4**).

The access adits will be excavated using DBT and will be used to allow access for assembling TBM 1, which will excavate the tunnel from the access adits to the tunnel inlet. The access adits will also be used for dismantling TBM 2, which will excavate from the tunnel outlet to the adits. The section of the tunnel between the access adits, as indicated in yellow in **Figure 4.5**, will be excavated using DBT.

Ventilation Shafts 1 and 2 will both be excavated using DBT. As indicated in Figure 4.5, the ventilation shafts will be positioned between the inlet and the access adits and between the access adits and the outlet, respectively.

The inlet and outlet portals will be excavated using DBT. As discussed in **Section 4.2.8**, the tunnel is excavated at a continuous slope of 0.027% from inlet to outlet.

The second phase of the project, to be implemented at a later date, involves the construction of a similar tunnel which is to be built alongside the tunnel discussed in this report. The first 100 m of the second tunnel will be excavated during the construction of the first tunnel. Thus, an additional excavation will take place at the inlet and this excavation will be done using DBT. The tunnel 2 inlet will be at an elevation of 940 masl.

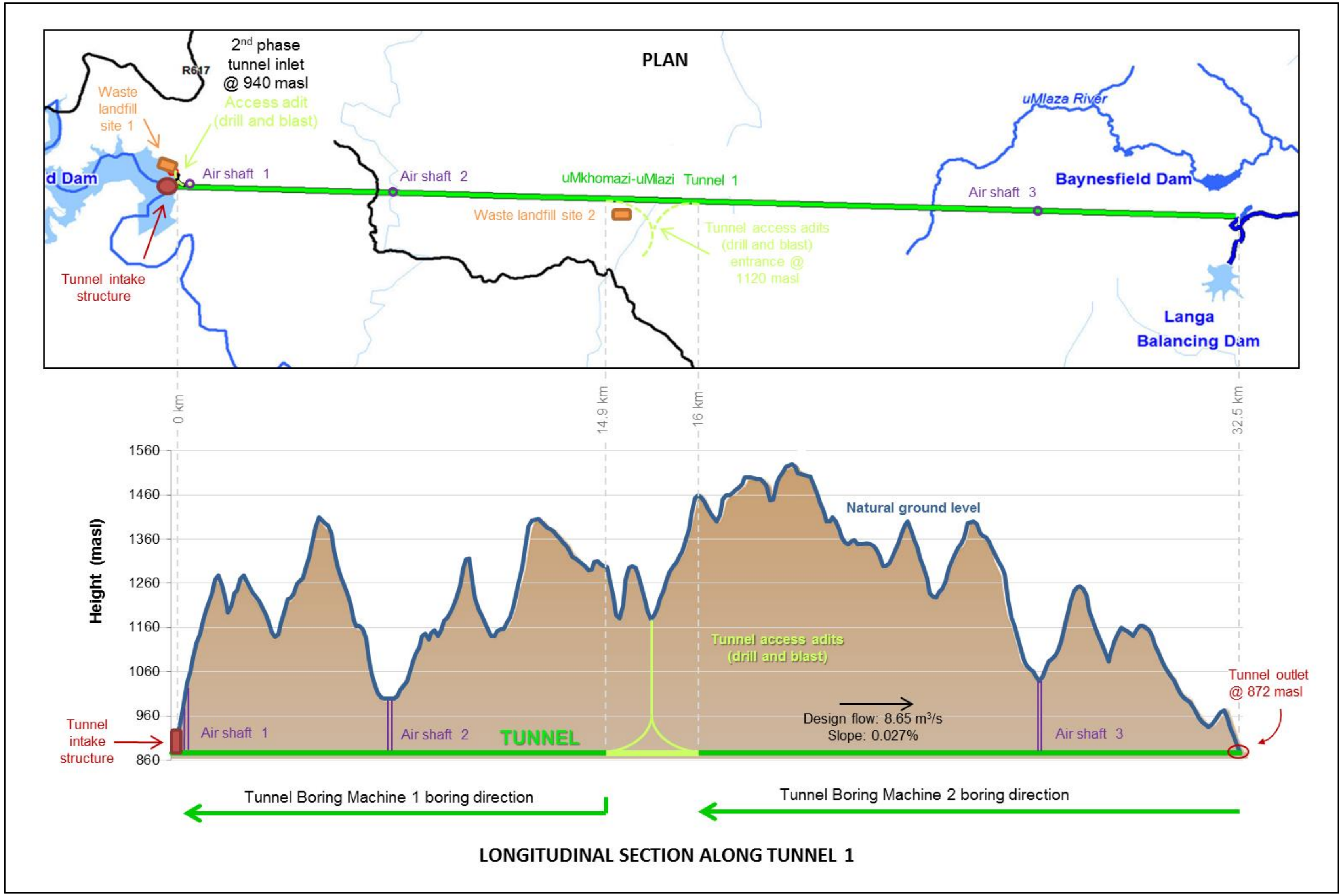


Figure 4.5: Selected tunnel layout

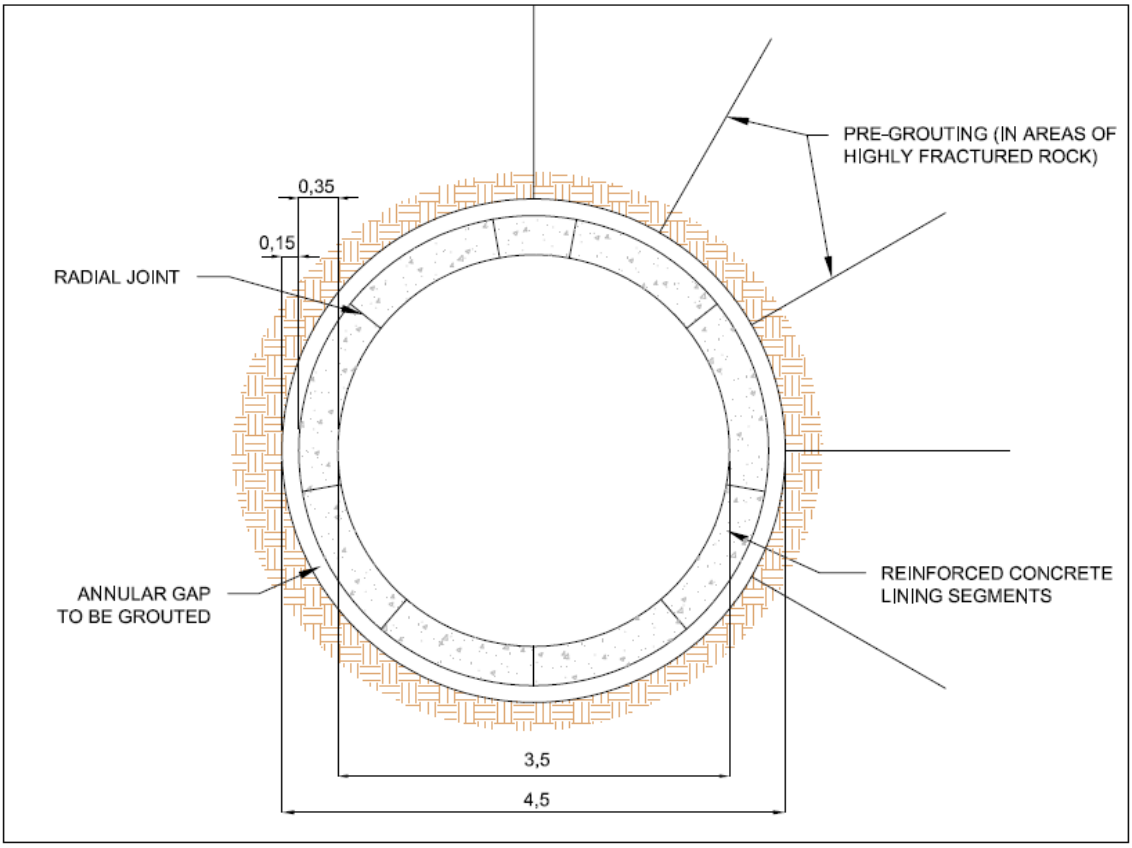


Figure 4.6: Cross section through tunnel

4.4.5 Tunnel spoil

The excavated material volumes which are to be removed from the tunnel, adits and ventilation shafts are shown in **Table 4.3**. Volumes are shown in bank cubic metre (BCM) and loose cubic metre (LCM). The LCM was based on a 1.6 swell factor.

Table 4.3: Excavated material volumes to be removed from tunnels

Tunnel Section	Excavated material, in-situ volume (BCM)	Excavated material (LCM*)
Tunnel 1 (portion from central adit to inlet portal)	233 014	372 823
Tunnel 1 (central tunnel section between adits)	32 558	52 093
Tunnel 1 (portion from outlet portal to central adit)	285 117	456 187
Tunnel 2 (first portion of tunnel)	1 590	2 545
Tunnel 1 central access adits	79 334	126 934
Tunnel 2 access adit	12 959	20 734

Tunnel Section	Excavated material, in-situ volume (BCM)	Excavated material (LCM*)
Ventilation Shaft 1	216	346
Ventilation Shaft 2	2 598	4 156
Ventilation Shaft 3	3 593	5 750

During the excavation of the tunnel it is expected that groundwater will be encountered. This too will have to be removed and, depending on the quality of the water, either pumped back into an existing water source, treated and then pumped back into an existing water source or stored in a lagoon.

4.5 TUNNEL INLET AND OUTLET PORTALS (STABILITY ANALYSIS OF PORTALS)

4.5.1 General

The materials and geotechnical investigation, carried out for the feasibility design of the Smithfield Dam tunnel inlet and Baynesfield tunnel outlet *(Geotechnical Report: Supporting Document 5)*, provides the required geological and geotechnical model for the design of the tunnel portals' excavation. The material properties of the soil and rock found at the position of the tunnel portals assist in determining how these materials will behave under excavation.

4.5.2 Portals designs and excavation volumes

The tunnel portal excavations are shown in **Figure 4.7** and **Figure 4.8**. The excavation material volumes are shown in **Table 4.4**. **Annexure 4 E** includes drawings of the tunnel excavations which show a plan and sectional view of the planned excavations for the inlet portal (**Figure 4.E.1**) and outlet portal (**Figure 4.E.2**).

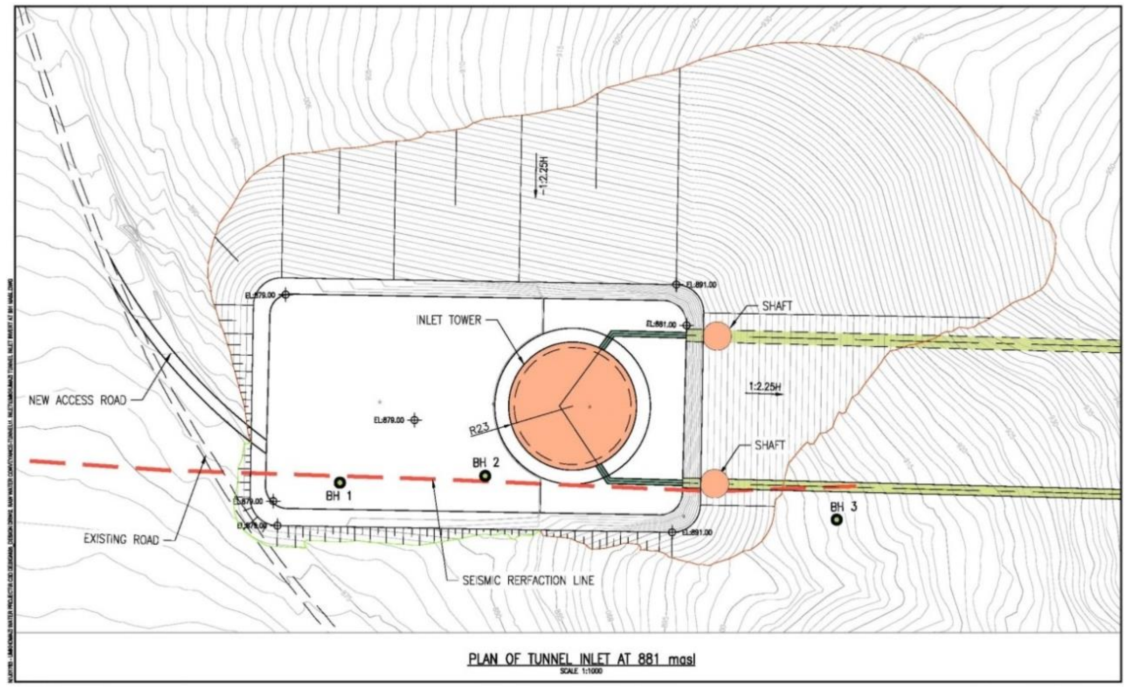


Figure 4.7: Tunnel inlet portal excavation

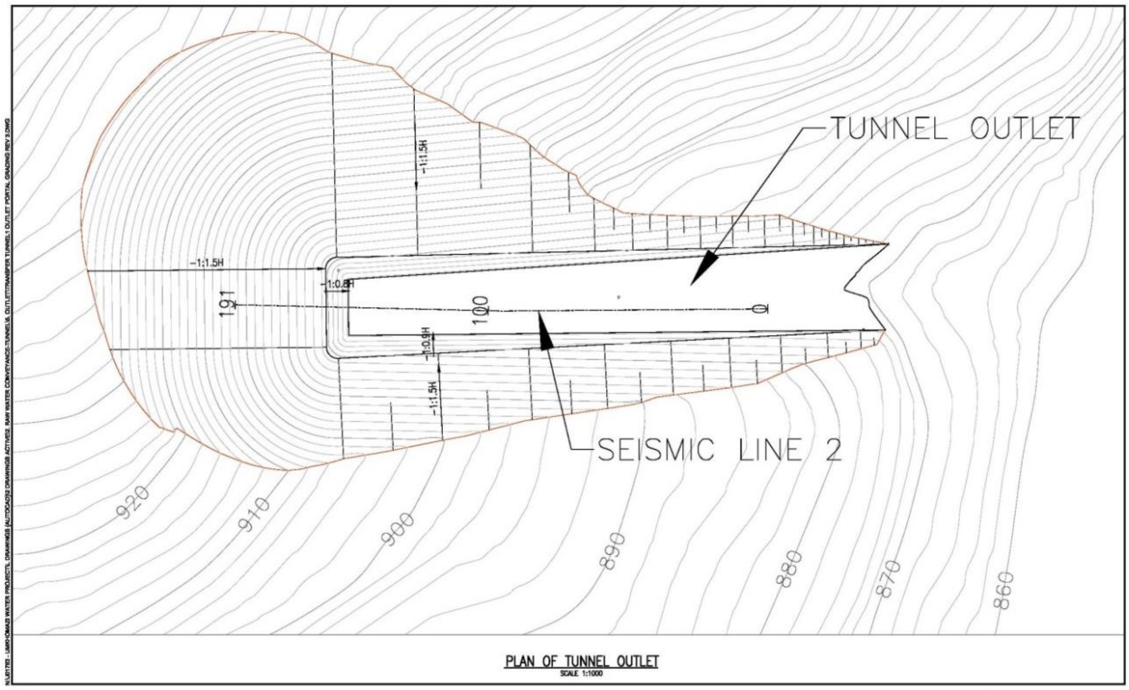


Figure 4.8: Tunnel outlet portal excavation

Table 4.4: Excavation volumes from tunnel portals

Portal	Excavated material, in-situ volume (BCM)	Excavated material (LCM)
Inlet	365 000	584 000
Outlet	401 000	641 600

4.5.3 Rock materials and geotechnical investigation

The tunnel traverses sub-horizontally bedded shale rock of the Karoo Sequence that has been extensively intruded by Dolerite. Both the shale and the dolerite are slightly to highly fractured, very hard to extremely hard rock. A layer of soil, with a thickness of roughly 6 m, overlays the hard shale rock. Groundwater was encountered close to ground surface at the inlet and outlet portals.

The properties of materials used in the analysis of both the inlet and outlet portal can be seen in **Table 4.5**.

Table 4.5: Material properties used in design software

Parameter	Shale Rock	Soil	Saturated Soil
Cohesion (kPa)	5 000	10	10
Phi (°)	35	34	34
Unit weight (kN/m³)	27.15	17	17

4.5.4 Slope stability criteria

In order to ensure a safe design against slip failure the slope must satisfy the criteria for steady-state seepage under operational conditions. The minimum factor of safety (FoS) required to yield a safe design in the permanent case is equal to 1.5.

In addition, at the inlet portal, the slope has to satisfy the safety criteria under rapid drawdown conditions. The minimum FoS required for rapid drawdown from FSL (930 masl) is 1.1 while that from emergency supply level (ESL) (938 masl) is 1.0.

4.5.5 Slope stability analysis and results

GeoSlope (SlopeW) computer software was utilised to analyse different scenarios of slope stability at the tunnel inlet and outlet portals. SlopeW allows the designer to analyse the excavation geometry and assess the possibility of slip failure occurring within the soil mass under different groundwater conditions. Due to the inherent strength of the shale rock being considerably greater than the soil material, slip failure is unlikely to occur in the hard shale rock. The configurations and results of each of the cases analysed for the inlet and outlet portals are attached in **Annexure 4 E** as **Figure 4.E.3** to **Figure 4.E.22**.

a) *Inlet portal*

Figure 4.9 indicates the position of two critical sections through the tunnel inlet portal slope. The SlopeW analysis was carried out for both Section 01 and Section 02. The final excavation design was, however, only based on Section 01 which proved to be the more critical of the two sections.

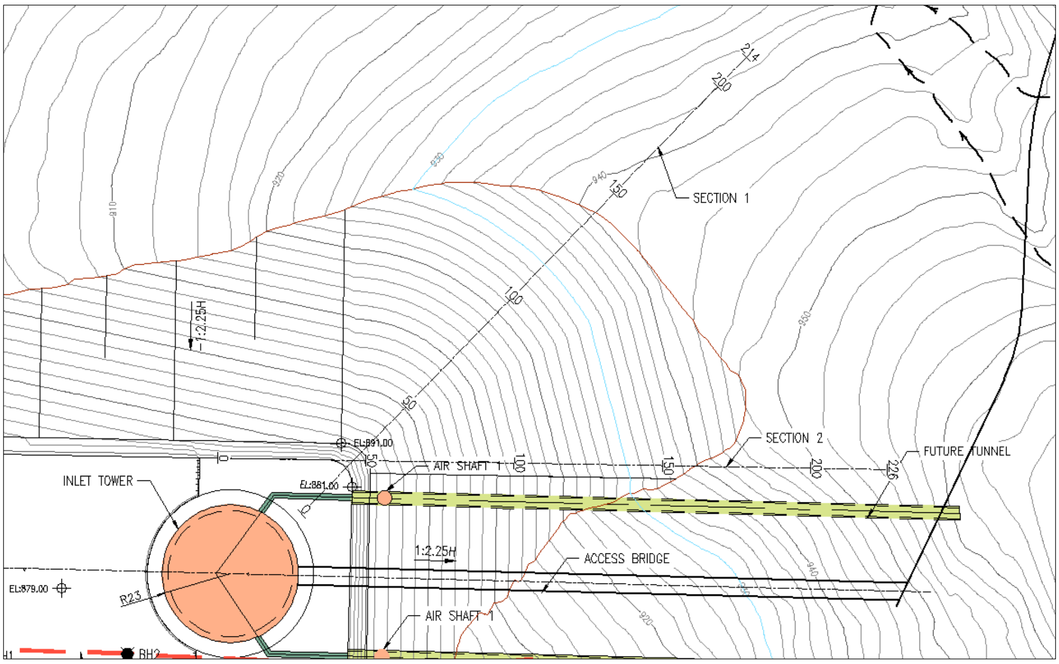


Figure 4.9: Plan view of the tunnel inlet portal showing Section 01 and Section 02

Figure 4.10 and **Figure 4.11** show the longitudinal sections of Section 01 and Section 02 respectively. It should be noted that the dashed line in both figures represents the natural ground level (NGL) and the solid line the initial excavation design. The initial excavation design for Section 01 and Section 02 yielded a FoS of 3.146 and 2.753, respectively. In both cases the FoS is greater than what is required to ensure a safe design against slip failure.

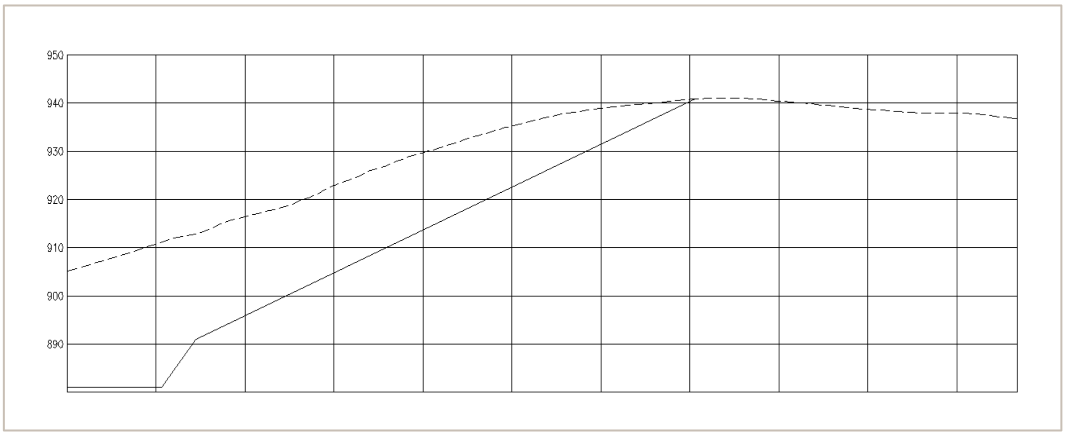


Figure 4.10: Longitudinal Section 01 at tunnel inlet embankment and initial excavation design

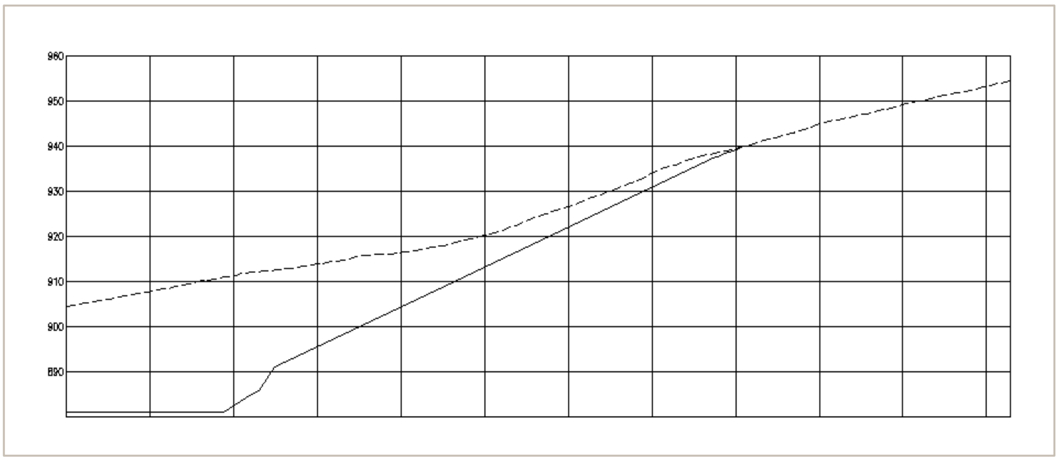


Figure 4.11: Longitudinal Section 02 at tunnel inlet embankment and initial excavation design

In order to optimise the design, various excavation methods were analysed. **Table 4.6** shows the different designs and corresponding FoS results obtained for Section 01.

Table 4.6: Results obtained for Section 01

Case	Description of excavation	FoS
1	90° angle from chainage 881 to top of saturated soil	0.756
2	90° angle from 881 to 885, slope 1:0.2 from 885 to top of saturated soil	0.877
3	90° angle from 881 to 885, slope 1:0.2 from 885 to top of shale rock, slope 1:2 in soil	3.036
4	90° angle from 881 to top of shale rock, slope 1:2 in soil	2.808

As expected, when a flatter slope was used for the excavation, a higher FoS was obtained. The rock mass exerts less force on the rock support and the FoS increases. Cases 1 and 2 yielded safety factors lower than what is required, revealing inadequate designs for the specific embankment. As for cases 3 and 4, the design yielded a FoS higher than what was required. Therefore an excavation design such as either case 3 or 4 should be used for Section 01.

Table 4.7 shows the results obtained for Section 02.

Table 4.7: Results obtained for Section 02

Case	Description of excavation	FoS
1	90° angle from chainage 881 to top of saturated soil	0.839
2	90° angle from 881 to 885, slope 1:0.2 from 885 to top of saturated soil	1.228
3	90° angle from 881 to 885, slope 1:0.2 from 885 to top of shale rock, slope 1:2 in soil	2.689
4	90° angle from 881 to top of shale rock, slope 1:2 in soil	2.801

As expected, the same cases yielded more or less the same results as with Section 01 in terms of safety factors. For example, a 90° excavation yielded a FoS lower than 1.5 and less significant angles of excavation led to higher FoS values. From these results it was clear that the best design was cases 3 and 4.

b) *Outlet portal*

The same method followed for the design of the tunnel inlet portal was used for the tunnel outlet portal. **Figure 4.12** shows the position of two critical sections through the tunnel outlet portal embankment.

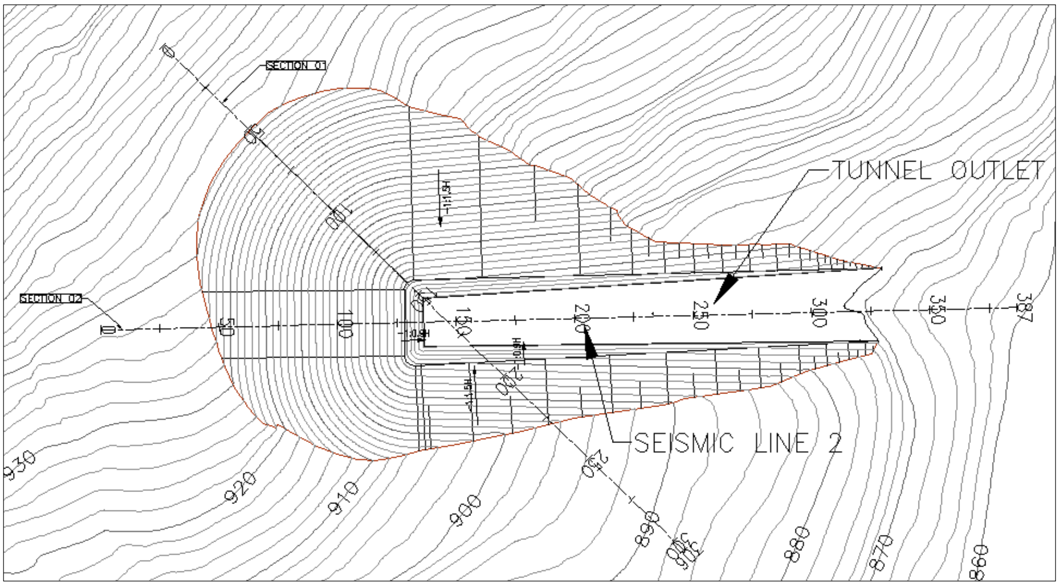


Figure 4.12: Plan view of tunnel outlet portal showing Sections 01 and 02

The material at the outlet portal is hard shale rock covered by roughly 6 m of dry soil, as opposed to the saturated soil at the inlet portal. The properties of the materials present are the same as those found at the inlet portal.

Figure 4.13 and **Figure 4.14** show Section 01 and Section 02, respectively, through the tunnel outlet portal embankment. It should be noted that the dashed line represents the natural ground level and the solid line the initial excavation design. The initial excavation design for Section 01 and Section 02 yielded a FoS of 2.603 and 2.720, respectively, which was greater than the FoS required against slip failure.

At the outlet portal, it was again expected that the slip failure would occur in the soil material rather than the hard shale rock. **Table 4.8** and **Table 4.9** show the results obtained for different cases analysed for the tunnel outlet portal for Section 01 and Section 02, respectively.

Table 4.8: Results obtained for Section 01

Case	Description of Excavation	FoS
1	90° angle from chainage 871.5 to top of soil	0.744
2	Slope 1:0.2 from 871.5 to top of soil	0.892
3	Slope 1:0.2 from 871.5 to top of shale rock, slope 1:2 in soil	2.177
4	90° angle from 871.5 to top of shale rock, slope 1:2 in soil	2.867

Table 4.9: Results obtained for Section 02

Case	Description of Excavation	FoS
1	90° angle from chainage 871.5 to top of soil	0.960
2	90° angle from 871.5 to 875.5, slope 1:0.2 from 875.5 to top of soil	0.977
3	90° angle from 871.5 to 875.5, slope 1:0.2 from 875.5 to top of shale rock, slope 1:2 in soil	2.463
4	90° angle from 871.5 to top of shale rock, slope 1:2 in soil	2.574

For cases 1 and 2, for Section 01 and Section 02, the obtained FoS was less than the required FoS which governs safe conditions. With the less steep slope of 1:2 in the soil material in cases 3 and 4, the rock mass exerts less force on the rock support and the FoS increases to over 3. Cases 3 and 4 therefore yielded an acceptable excavation design for the tunnel outlet portal.

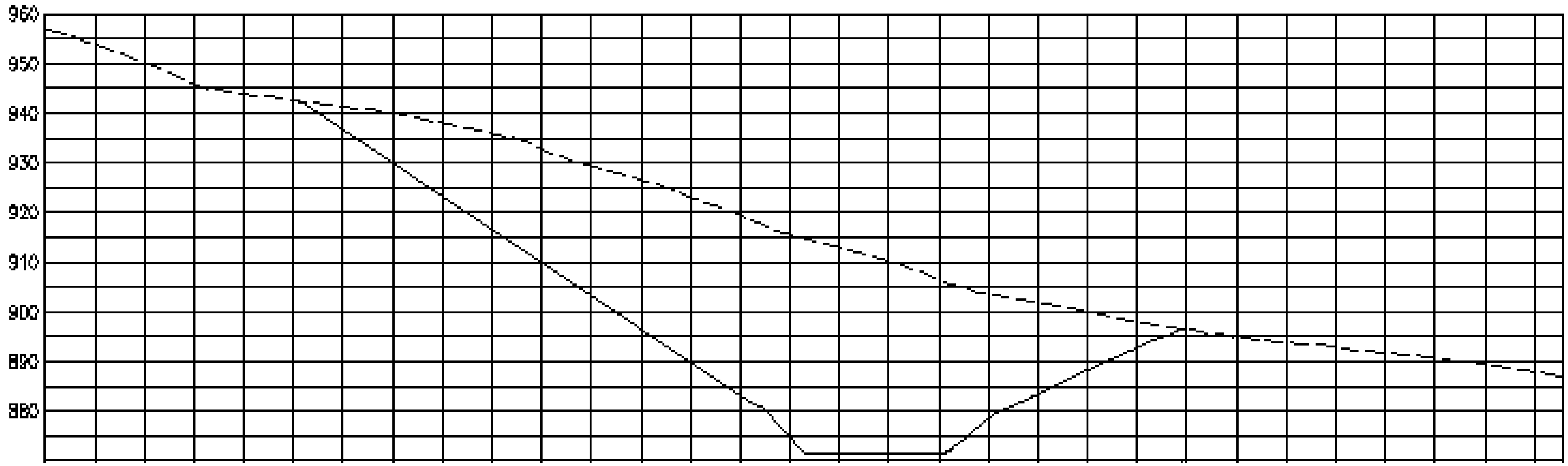


Figure 4.13: Embankment longitudinal section and initial excavation design for Section 01

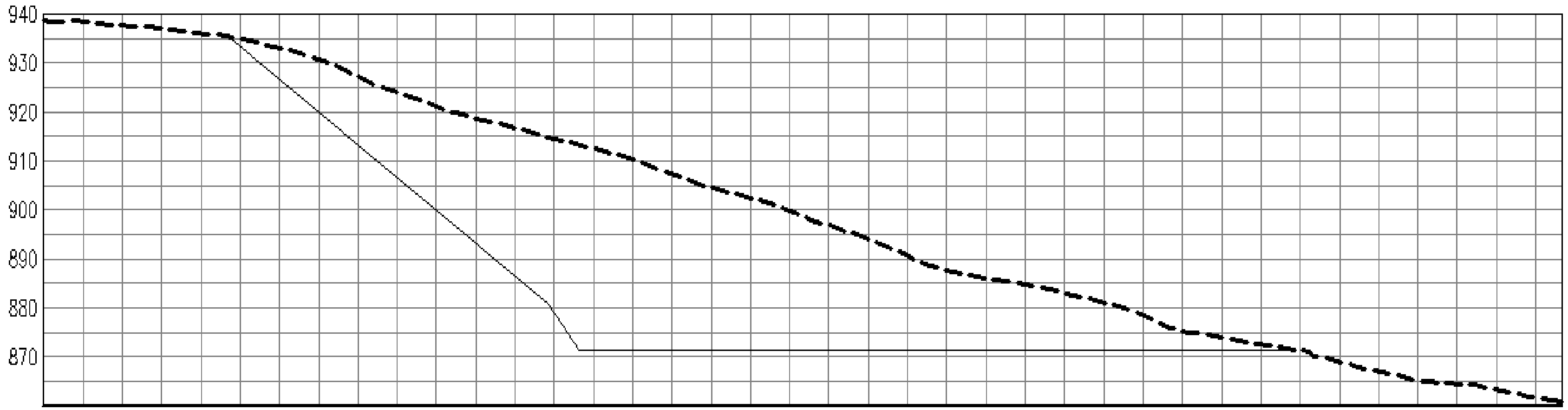


Figure 4.14: Embankment longitudinal section and initial excavation design for Section 02

4.5.6 Practical construction considerations

Even though the design software indicated that a 90° excavation angle is sufficient and safe, it was recommended that, for practical purposes, both the inlet and outlet portal be excavated in a step formation at a slope of 1:1.5. Therefore, the height and width of each step formed during blasting is 2 and 3 m, respectively. Excavating at a slope of 1:1.5 is the most practical and safe design for the tunnel portals.

4.6 TUNNEL OUTLET STRUCTURE AND ACCESS ARRANGEMENTS

During the design of the tunnel outlet in the Baynesfield valley, the visual impact of the tunnel outlet was minimised. A preliminary design of the tunnel outlet arrangement for the following components was undertaken:

- an open area with a concrete slab for the assembling of the TBM during construction;
- the transition section between the tunnel and pipeline;
- the mass concrete unit used for the submergence of the pipeline underground; and
- access into tunnel for future maintenance.

Drawings of this preliminary design are contained in **Annexure 4 G**.

The concrete slab (16 m x 50 m) for assembling of the TBM during construction will not be required after the construction phase, and should be decommissioned once redundant. However, a short section (approximately 6 m wide and 30 m long) should be left for access to the outlet structure (see **Section 4.6.4**). The area of the removed slab should be rehabilitated. The preliminary layout of the access arrangements should be reviewed in the detail design phase.

A mass concrete structure consisting of two units is required, firstly for the transition between the 3.5 m diameter concrete tunnel and the 2.6 m diameter steel pipeline; and secondly for the vertical change in the level of the steel pipeline. Access into the tunnel is required during the operation of the scheme and forms part of **Room 1**. Both rooms have a minimum of 2 m concrete cover surrounding the pipeline to ensure stability of the conveyance system and will be covered with soil and rehabilitated with grass to minimise the visual impact.

4.6.1 Mass Concrete Room 1

a) Access to main tunnel

Vehicular access to the inside of the tunnel at the outlet structure is required to traverse the tunnel along its length during inspection. Two alternatives were considered, taking into account hydraulic functionality of the tunnel, cost and aesthetics.

- Option 1

The first option was a short section of tunnel (included in **Room 1**) that would provide access to the tunnel at the outlet portal. The end of this section would be closed with a steel door of 3.8 m by 3.8 m, which would only be opened for inspection once the tunnel has been drained. A portion of the length of the driveway up to the access point will be covered by pre-cast box culverts to ensure the proper covering of the concrete housing units with soil.

- Option 2

The second option was an access adit of 600 m in length, entering the tunnel just upstream of the outlet portal. It would daylight at an elevation above the FSL of Smithfield Dam so that it can remain open during operation of the tunnel. To be able to drive in the adit, a grade of 1V:10H would be needed, which would result in an adit approximately 600 m long. This length of drill and blast construction would be very high in comparison to the short section of tunnel and steel door required for the first option.

Both options would allow the tunnel to function normally, with a similar minimal aesthetic impact.

Table 4.10 and **Table 4.11** show the summary of the estimate cost for each option, respectively.

Table 4.10: Estimated cost for access at tunnel outlet portal (Option 1)

Description	Unit	Qty	Rate (R)	Total cost (R)
Concrete				
(a) Room 1	m^3	1163	2 500	2 907 500
(b) Room 2	m^3	1058	2 500	2 645 000
Formwork				
(a) Horizontal	m^2	136	550	74 800
(b) Vertical	m^2	1056	550	580 800
Reinforcement				
(a) Room 1	t	116.3	10 000	1 163 000
(b) Room 2	t	105.8	10 000	1 058 000
Pre-cast box culverts	No.	30	15 000	450 000
Total cost				**8 879 100**

Table 4.11: Estimated cost for access adit (Option 2)

Description	Unit	Qty	Rate (R)	Total cost (R)
Adit Excavation	m^3	13 400	2 230	29 882 000
Rockbolts	m	7 458	285	2 125 530
Shotcrete	m^3	234	5 885	1 377 090
Reinforcing mesh	m^2	2 084	85	177 140
Concrete	m^3	827	2 500	2 067 500
Formwork	m^2	32 044	550	17 624 200
Grouting	m^2	32 044	485	15 541 340
Total cost				**68 794 800**

From **Table 4.10** and **Table** 4.11 the estimated cost for Option 1 is approximately R 9 million, while the estimated cost for Option 2 is approximately R 68.8 million. Therefore, Option 1 was selected as the feasible option for the tunnel outlet structure and should be considered for detail design (see **Figure 4.G.2**).

b) Convergence of the tunnel to the pipeline

Room 1 consists of the interface required to change the diameter of the conveyance system from 3.5 m for the concrete tunnel to 2.6 m for the steel pipeline. The transition of the pipes should be tapered and smoothed to limit hydraulic pressure losses and housed in mass concrete to minimise the negative visual impact. The concrete will also be covered with soil and rehabilitated with grass. The total length for the converging section will be

12 m to provide stability to the structure and ensure a smooth transition for the conveyance system.

4.6.2 Mass Concrete Room 2

Room 2 will be required to divert the pipeline from being above the natural ground level to being submerged at a level of 1.5 m below the natural ground level. Krüger (1977) suggested that a radius of between 3 to 7 times the open channel width should be used for the deviation of the vertical alignment of the pipeline. This resulted in an approximate radius of between 10 and 24 m for the pipe bend. These values serve only as guidance due to the difference between open channel flow and conduit flow conditions. A radius of 20 m was selected to create more favourable flow conditions with less losses, resulting in a total length of 13 m for the vertical deviation of the steel pipeline (see **Figure 4.G.2**).

4.6.3 Valve chamber

Although not directly related to the visual impact of the outlet infrastructure, the positioning of a valve at the start of the pipeline was considered during this assessment due to the impact it may have on the layout. The reason a valve is required at the start of the pipeline is to allow for inspection of the pipeline without having to drain the tunnel. The only specification is that the valve should not be closer than a distance of 6 diameters of the pipe size, downstream of a bend, for ideal functionality of the valve. Therefore, the valve should be approximately 16 m downstream of the bend where the pipe is submerged. The final position of the valve should be considered during the detail design phase. The valve chamber would be according to typical details from standard DWS drawings.

4.6.4 Access to tunnel

Maintenance vehicles are required to enter the tunnel through **Room 1** as discussed in **Section 4.6.1**. Pre-cast box culverts should be aligned to form a drive-way tunnel, ensuring permanent access for maintenance vehicles. The short section of concrete left in place after assembling the TBM, will form the foundation for the box culverts. A steel security door should be placed at the entrance of the drive-way tunnel to prevent unauthorised access. The visual impact of the tunnel outlet structure will be minimised by backfilling the area surrounding the box culverts and rehabilitate the area with grass.

4.6.5 Artistic impression of tunnel outlet structure

An artistic impression of the tunnel outlet structure was produced to portray the visual impact of the outlet structure on the surrounding landscape. **Figure 4.15**, **Figure 4.16** and **Figure 4.17** indicate the expected visual impact of the tunnel outlet at Baynesfield valley from different viewing angles.

As seen in the figures below, the visual impact was minimised by rehabilitating the excavated area and covering the concrete rooms with soil and grass to match the natural surrounding area.

Figure 4.15: Three dimensional artistic impression of the tunnel outlet in the Baynesfield valley - View 1

Figure 4.16: Three dimensional artistic impression of the tunnel outlet in the Baynesfield valley - View 2

Figure 4.17: Three dimensional artistic impression of the tunnel outlet in the Baynesfield valley - View 3

4.7 SUMMARY OF TUNNEL COMPONENTS AND COST ESTIMATES

A summary of the tunnel, shafts and adits components are shown in **Table 4.12** and the estimated costs associated with implementing these components have been included in **Table 4.13**. A detailed cost estimate of all construction activities, comprising quantities and rates, has been completed, and is contained in **Annexure 4F**. Assumptions made in determining all cost estimates are described in **Section 14**. The total scheme cost estimate with all components added together is given in **Section 14.4**.

Table 4.12: Excavation and lining details for all tunnel components

Tunnel Component	Excavation Details			Tunnel Lining Lengths	
	Method	Outside Diameter (m)	Finished Internal Diameter (m)	Shotcrete Lined (m)	Concrete Lining (m)
Tunnel 1 (portion from central adit to inlet portal)	TBM	4.50	3.50		14 651
Tunnel 1 (portion from outlet portal to central adit)	TBM	4.50	3.50		17 927
Tunnel 2 (first portion of tunnel)	D & B	4.50	3.50	100	
Tunnel 1 central access adit (central tunnel section)	D & B	5.25	5.00	1 504	
Tunnel 1 central access adit (combined shaft)	D & B	5.25	5.00	706	
Tunnel 1 central access adit (TBM 1 entrance shaft)	D & B	5.25	5.00	1 181	
Tunnel 1 central access adit (TBM 2 exit shaft)	D & B	5.25	5.00	1 190	
Tunnel 2 access adit	D & B	5.25	5.00	500	
Ventilation Shaft 1	D & B	5.25	5.00	10	
Ventilation Shaft 2	D & B	5.25	5.00	120	
Ventilation Shaft 3	D & B	5.25	5.00	166	

Table 4.13: Summary of cost estimate of all activities for the uMkhomazi – uMlaza Tunnel

Description	Cost (R million, excl. VAT)
Tunnel 1	2 664.1
Tunnel 1 inlet portal	202.7
Tunnel 1 outlet portal	208.9
Tunnel 1 central access adits	205.3
Tunnel 1 adit portal	41.4
Tunnel 1 ventilation shafts	7.2
Tunnel 2	5.6
Tunnel 2 inlet portal	1.4
Tunnel 2 access adit	25.6
Miscellaneous	539.1
Total	**3 901.2**

4.8 RECOMMENDATIONS

Further geotechnical investigations are to be carried out during the tender design phase to assess tunnel conditions, the need for lining, and groundwater conditions, including quality aspects.

4.9 REFERENCES

AECOM, AGES, MMA & Urban-Econ, 2014. *The uMkhomazi Water Project Phase 1: Module 1: Technical Feasibility Study: Raw Water; P WMA11/U10/00/3312/3/1/1 - Supporting document 1: Optimisation of conveyance system report*, Pretoria, South Africa: Department of Water Affairs (DWA).

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Kruger, N., 1977. *Toegepaste Hidrolika 5SW: Kanaalontwerp,* Pretoria, Universiteit van Pretoria: Departement Siviele Ingenieurswese

5 LANGA DAM

5.1 SIZING BACKGROUND

5.1.1 Criteria

Langa Dam is required to store water for the supply of raw water to the planned Umgeni Water's Baynesfield WTP during maintenance periods and emergencies of the uMkhomazi – uMlaza Tunnel, when raw water cannot be supplied directly from Smithfield Dam. The criteria applied for the sizing of Langa Dam are the following:

- Two months of supply should be available. If the storage that can be provided is not sufficient, the maximum possible storage should be provided and the rest can be provided from other sources through the integrated Mgeni WSS.
- The FSL of Langa Dam is limited by the FSL of Smithfield Dam (930 masl), since Langa Dam has to be filled under gravity from Smithfield Dam.
- The hydraulic requirements of supplying water through the tunnel to Baynesfield WTP and for filling Langa Dam, and thereby accommodating friction and secondary losses, must be met. During off-peak periods, and when Smithfield Dam reservoir is at high levels or spilling, raw water will be supplied to both the WTP and Langa Dam for storage.

5.1.2 Hydraulic requirements

Langa Dam will be supplied under gravity from Smithfield Dam via the uMkhomazi – uMlaza Tunnel and pipeline. The estimated total friction and secondary losses from the tunnel outlet up to Langa Dam is 6.98 m and is for the following components of the scheme:

- The 32.5 km long 3.5 m diameter uMkhomazi – uMlaza Tunnel;
- The 0.4 km long part of the 2.6 m diameter pipe from the tunnel outlet up to the take-off to Langa Dam; and
- The 1.52 km long 1.6 m diameter pipeline from the take-off to Langa Dam.

For the estimated total losses of 6.98 m over the aforementioned components, Langa Dam's FSL cannot exceed 923.02 masl and therefore the recommended FSL for the dam is 923 masl.

5.1.3 Summary of sizing

Langa Dam's FSL cannot exceed 923 masl given the hydraulic requirements and a full Smithfield Dam. The live storage that therefore can be achieved in Langa Dam is about 14.82 million m³, taking the additional storage created by the quarry in the dam basin into account – see **Table 5.1** and **Table 5.24**. This volume represents about 24 days of supply, at an average flow rate of 7.10 m³/s to the Baynesfield WTP, with no water supply from the tunnel. The remainder of the required two months of supply will have to be provided from the existing Mgeni WSS. This option is described in detail in the relevant *Water Resources Planning Model Report (P WMA 11/U10/00/3312/2/4)*.

5.2 OPERATION RULE FOR LANGA DAM

In accordance with the water yield analysis carried out, the following operation rule was developed for the operation of Langa Dam:

- Langa Dam is to be filled and supported by Smithfield Dam when Smithfield Dam is spilling.
- Langa Dam is to release water for EWR between the dam and the new Mbangweni Dam.
- Langa Dam is to provide water to the Baynesfield WTP during maintenance and repair periods of the tunnel.

5.3 AREA – STORAGE VOLUME CHARACTERISTIC

The area-capacity tables for Langa Dam, for the scenarios without and with the quarry, are given in **Table 5.1**. The proposed MOL for Langa Dam is 898.24 masl; the volume between the FSL and MOL can then supply the WTP for a period of three weeks and three days if the additional storage capacity that is created by the quarry is also taken into account. The balance table for the materials is given in **Section 5.4** of this report.

Table 5.1: Area capacity tables for Langa Dam

Contour (masl)	Surface Area (ha)		Gross Storage (million m³)	
	Without Quarry	With Quarry	Without Quarry	With Quarry
880	0.54	0.54	0.00	0.00
885	3.08	3.08	0.09	0.09
890	7.40	7.40	0.35	0.35
895	13.28	13.28	0.87	0.87
898	19.95	26.45	1.43	1.43
900	24.39	30.33	1.81	1.87
905	36.25	41.22	3.33	3.53
910	49.84	51.13	5.48	5.82
915	66.51	67.30	8.39	8.88
920	83.95	84.13	12.15	12.78
923	**95.41**	**95.48**	**14.95**	**15.67**
925	103.05	103.05	16.82	17.59
930	120.42	120.42	22.41	22.41
935	139.31	139.31	28.91	28.91

5.4 FOUNDATION AND CONSTRUCTION MATERIALS

The area around the proposed site is underlain by rocks of the Pietermaritzburg Formation of the Ecca Group, comprising shales and siltstones with subordinate sandstones. One near-horizontal dolerite sill had intruded concordantly into the sedimentary strata.

Due to the presence of an extensive wetland in the dam basin and of cultivated lands on the left flank, environmental restrictions were placed on seismic surveys, test pits and boreholes in certain areas.

Seismic refraction surveys were conducted across the dam centre line. Although the seismic velocities tended to over-estimate the depth of sound rock, they were useful in showing the presence of the dolerite sill below a cover of shale and also to identify the position of a fault.

The following four potential sources for construction materials were investigated:

- Spoil from the tunnel excavation;
- Material excavated from the tunnel outlet portal;
- Material excavated from the spillway approach area; and
- Material from a quarry located below FSL in the dam basin.

It appears that none of the available materials qualify as impervious fill, and that the available quantity of semi-pervious material is not quite sufficient to provide twice the required volume for a zoned embankment dam.

A considerable volume of soft rockfill (weathered shale) will have to be removed from the quarry in order to reach the underlying hard shale and dolerite for rockfill. This soft material can be used in certain zones of any of the alternative embankment dam types.

There is, however, sufficient hard shale rockfill available for the construction of a concrete faced rockfill (CFR) dam or earth core rockfill (ECR) dam. For any of these dam types, durable dolerite may have to be imported from a commercial quarry (Pietermaritzburg) to serve as a protective layer above the shale or dolerite/shale mixture from the tunnel excavation and the quarry. However, it is possible that some or all of this dolerite might be obtained from the quarry, but this will require further investigation.

Spoil from the tunnel excavation is expected to have the properties of G5 gravel and can be compacted to form part of a rockfill embankment. Due to the absence of impervious core material, two dam types were considered; namely a CFR dam and a CCR dam with impervious core comprising a mixture of soil and bentonite. The CFRD option, however, appears to be the most feasible, since the available soils contain too much clay to mix with bentonite.

For the shells of the proposed rockfill embankment, between 1.6 and 5.3 m of colluvium and residual soil/completely weathered shale will have to be removed along most parts of the centre line. However, in an area on the right flank, weak completely weathered shale and dolerite extend to a depth of over 17 m and will have to be removed.

It will be necessary to make provision for a grout curtain to a depth of about 66% of the water head along the centre line.

The spillway control structure and chute can be founded on moderately weathered shale at an average depth of about 5.5 m.

The risk for slope failures around the rim of the reservoir that might endanger the dam wall is considered negligible.

As a result of environmental restrictions on the positioning of seismic lines, test pits and boreholes, this investigation was limited.

Table 5.2: Estimated volumes of embankment materials from borrow areas, tunnel spoil and the tunnel outlet portal for CFR dam with FSL of 923 masl

Source	Type of construction material available							
	Overburden for spoil	Impervious core	Semi-pervious fill	Semi-pervious fill	Soft rockfill	Hard rockfill	Imported Dolorite	Total available (m³)
	Organic topsoil	Clayey sand transported material	Residual silty clayey sand and sandy silty clay	Highly weathered shale	Moderately weathered shale	Unweathered shale and dolerite		
Quarry I	20 000	-	120 000	180 000	350 000	1 200 000	-	**1 870 000**
Portal Excavation	8 000	-	230 000	70 000	50 000	40 000	-	**398 000**
Tunnel Spoil	-	-	-	-	-	250 000	-	**250 000**
Spillway Approach	15 000	-	35 000	280 000	20 000	-	-	**350 000**
Dam Excavation	138 261	-	-	182 516	182 516	212 936	-	**716 229**
Other	-	-	-	-	-	-	-	**-**
Total available	**181 261**	**-**	**385 000**	**712 516**	**602 516**	**1 702 936**	**-**	**3 584 229**
Dam component								
Hard Rockfill	-	-	-	-	-	620 236	-	**620 236**
Hard Rockfill (Tunnel Spoil)	-	-	-	-	-	250 000	-	**250 000**
Gravel Layers (Upstream)	-	-	-	-	-	-	41 386	**41 386**
Concrete	-	-	-	-	-	8 973	-	**8 973**
Downstream protection layer	-	-	-	-	-	-	144 615	**144 615**
Downstream Berm	-	-	230 000	70 000	50 000	212 936	-	**562 936**
Total required	**-**	**-**	**230 000**	**70 000**	**50 000**	**1 092 145**	**186 001**	**1 628 146**
Balance	**181 261**	**-**	**155 000**	**642 516**	**552 516**	**610 791**	**-186 001**	**1 956 083**
Percentage Remaining	**100%**	**-**	**40%**	**90%**	**92%**	**36%**	**-**	**55%**

Additional test pitting, core drilling, sampling and laboratory tests are recommended during the design stage in order to confirm the properties and volumes of construction materials actually required and to confirm founding conditions for the selected type of dam and spillway structure.

The estimated available material volumes for the proposed CFR dam are summarised in **Table 5.2**.

5.5 DAM TYPE SELECTION

The dam type selection is described in the *Dam Type Selection Report (Engineering Feasibility Design Report: Supporting Document 5)*. The dam types indicated in **Table 5.3** have been considered. Reasons for not considering the dam type are also indicated.

Table 5.3: Dam type options considered

Dam type	Reason for not selecting the dam type as indicated
Roller compacted concrete (RCC) gravity dam	Selected
Concrete faced rockfill dam (incl. various options of zoning depending on availability of material)	Selected
Composite dam (various options of gravity dam with any of the above-mentioned embankment dams)	Selected
Earth core rockfill dam (incl. various options of zoning depending on availability of material)	• Insufficient impervious material and semi-pervious material
Zoned earthfill embankment dam	• Insufficient impervious material and semi-pervious material
Conventional vibrated concrete (CVC) gravity dam	• More expensive (with a higher cement content) than RCC gravity dam
Conventional vibrated concrete (CVC) buttress dam	• More expensive than both RCC and CVC gravity dams • Time-consuming
Concrete arch dam	• More expensive than both RCC and CVC gravity dams • Valley shape not favourable
Hardfill concrete gravity dam	• More expensive than both RCC and CVC gravity dams • Would need too much aggregate that is not necessarily available on site
Asphalt concrete gravity dam	• Too expensive • Earthfill materials for the core (more favourable than asphalt) are available on site
Masonry/hand labour intensive methods	• This type is expensive and is therefore not recommended for implementation.

As can be seen from **Table 5.3**, based on the available materials and making maximum use thereof, the following types were considered:

- CFRD
- RCC gravity
- Composite dam: central RCC gravity type with CFRD on left and right flank.

A summary of the cost estimation for these options is shown in **Table 5.4**.

Table 5.4: Cost estimates for dam types

Option	Dam type	Cost (R, excl. VAT)
1	CFRD	549 087 699
2	RCC gravity	1 591 187 651
3	Composite comprising an RCC central spillway section and CFRD left and right flank	1 148 067 443

Due to the lack of sufficient earthfill materials and relatively deep foundations encountered, the best dam type identified was a CFRD. This dam type also provided the least amount of material that would need to be spoiled.

5.5.1 Optimum use of materials

The sources of materials for Langa Dam are as follows:

- Bored shales and dolerites from the tunnel;
- Completely weathered shale materials from the tunnel exit portal;
- Shale materials from the quarry in the dam basin of Langa Dam; and
- Some weathered materials from the spillway approach channel.

A CFRD will be constructed with zones and materials in the zones as follows:

- Body of dam – shale rock from the quarry in the dam basin;
- Downstream toe of dam – bored rock from tunnel; and
- Downstream berm – completely weathered rock from the tunnel portals.

This option meets the requirements of making maximum use of the available materials as well as accommodating spoil materials from the tunnel without providing for a separate spoil landfill site for this purpose.

5.6 SELECTED DAM LAYOUT

In accordance with the *Dam Type Selection Report* a CFRD is proposed for Langa Dam. For the proposed NOC of 926.60 masl (see **Section 5.13**), the maximum wall height and width will be 46.60 m and 202.72 m respectively. The proposed embankment crest width is 7 m and the proposed upstream and downstream slopes of the rockfill embankment are 1V:2H and 1V:2.2H. The estimated total length of the dam wall is 573 m. The dam will inundate an area of about 95.48 ha at the proposed FSL of 923 masl, which is about 17.91% of the dam's catchment area.

A 10 m long ogee spillway on the left flank of the dam, with an approach channel with an ogee weir of 1.5 m depth, is proposed together with a 177 m long chute and stilling basin at the end. An inlet/outlet structure comprising one tower with a dual pipe system is proposed to serve the following purposes:

- An inlet structure for water that will be supplied to Langa Dam from Smithfield Dam;
- An outlet structure which will serve as the outlet for raw water supply to the WTP; and
- An outlet structure for water releases from Langa Dam for environmental purposes.

The layout of Langa Dam is included in **Annexure 5 A** as **Figure 5.A.1**.

5.7 FLOODS AND FLOOD HYDROGRAPHS

5.7.1 Hydrological analysis

The key hydrological parameters for the proposed Langa Dam are summarised in **Table 5.5**.

Table 5.5: Key hydrological parameters for Langa Dam

Catchment parameter	Value and unit
Catchment area	5.34 km²
Longest water course	2.65 km
Distance to catchment centre of catchment (L_c)	1.5 km
Average catchment slope (S)	0.0528 m/m
Mean annual precipitation (MAP)	931 mm/annum
Time of concentration (T_c)	0.40 hours

The peak flows at Langa Dam for the various recurrence intervals were estimated with the following methods:

- Rational method (since catchment is smaller than 15 km²);
- Alternative rational method;
- Unit hydrograph method;
- Standard design flood method;
- Empirical methods;
- TR137 method (RMF); and
- Statistical methods, based upon the available 16 year flow record at DWA Gauging Weir Number U1H003 on the Mkomazi River at Umkomazi Drift.

For the purposes of the statistical methods the recorded peak flows at Gauging Weir U1H003 were adjusted as follows for the statistical analysis:

$$Q_{\mathrm{Langa}} = Q_{\mathrm{U1H003}} \sqrt{\frac{A_{\mathrm{Langa}}}{A_{\mathrm{U1H003}}}} \qquad \textbf{(Equation 5-1)}$$

Where:

Q = flood peak discharge (m³/s)

A = catchment area (km²)

A_{Langa} = 5.34 km²

A_{U1H003} = 4 375 km²

The estimated peak discharges for Langa Dam with the various methods are summarised in **Table 5.6**.

Table 5.6: Estimated peak flows for Langa Dam

Method	Peak flows (m³/s)							
	Recurrence Interval (years)							
	2	5	10	20	50	100	200	RMF
Rational method	23	33	44	58	79	102	133	-
Alternative rational method	27	48	67	88	116	141	159	-
Unit hydrograph method	18	31	46	64	98	136	194	-
Standard design flood method	11	47	82	121	180	230	284	-
Empirical methods	-	-	46	63	87	110	142	-
TR 137 method	-	-	-	-	126	157	187	283
Statistical methods	22	34	48	61	82	101	125	-

5.7.2 Assessment of the estimated peak flows

The aforementioned methods for estimating peak flows at Langa Dam yielded a wide spread of results for the various recurrence intervals, as shown in **Table 5.6**. None of the methods could be recommended and therefore the following two approaches were followed, except for the RMF and SEF:

- **Approach 1:** Eliminate the lowest and highest values for each recurrence interval and calculate the average of the values in-between.
- **Approach 2:** Calculate the average of all the methods for each recurrence interval.

The results of these two approaches are summarised in **Table 5.7**. Approach 2, however, yielded the more conservative results.

Table 5.7: Estimated peak flows for Langa Dam with approaches 1 and 2

Approach	Recurrence interval (years)							
	2	5	10	20	50	100	200	RMF
	Estimated peak flows (m³/s)							
Approach 1	21	39	46	61	102	136	171	283
Approach 2	20	39	56	76	128	163	204	

5.7.3 Recommended peak flows, flood volumes and SEF for Langa Dam

The recommended peak flows and estimated flood volumes for Langa Dam are summarised in **Table 5.8** and **Table 5.9** respectively. The flood volumes were derived from a triangular shaped inflow hydrograph. Langa Dam is located in

Kovacs Region K7 for the purposes of the RMF calculation. The recommended SEF for Langa Dam, in terms of the SANCOLD Guidelines, is RMF + Δ, which is the equation for the RMF for Kovacs Region K8.

Table 5.8: Recommended peak flows for Langa Dam

Recurrence interval (years)								
2	5	10	20	50	100	200	RMF	SEF
Recommended peak discharges (m^3/s)								
20	39	56	76	128	163	204	283	313

Table 5.9: Estimated flood volumes for Langa Dam

Recurrence interval (years)								
2	5	10	20	50	100	200	RMF	SEF
Estimated flood volumes (million m^3)								
0.043	0.084	0.121	0.164	0.276	0.352	0.441	0.611	0.676

5.7.4 Inflow hydrograph for Langa Dam

Since Langa Dam's catchment area is only 5.34 km^2, a triangular shaped inflow hydrograph that peaks at the time of concentration (T_c) with a base of three times T_c is proposed. For flood routing purposes a base flow of 8.65 m^3/s was assumed for the scenario when 8.65 m^3/s continues to flow into Langa Dam from Smithfield Dam together with a flood event. The inflow hydrographs that were used for flood routing calculations, for the purposes of the spillway design and assessments are given in **Table 5.10.**

Table 5.10: Inflow hydrographs* into Langa Dam for flood routing calculations

Time (hours)	Inflow (m^3/s)		
	1:200 year	SEF	2 x RMF
0	8.65	8.65	8.65
0.1	59.65	86.90	150.15
0.2	110.65	165.15	291.65
0.3	161.65	243.40	433.15
0.4	212.65	321.65	574.65
0.5	187.15	282.53	503.90
0.6	161.65	243.40	433.15
0.7	136.15	204.27	362.40
0.8	110.65	165.15	291.65
0.9	85.15	126.03	220.90
1	59.65	86.90	150.15

Time (hours)	Inflow (m^3/s)		
	1:200 year	SEF	2 x RMF
1.1	34.15	47.77	79.40
1.2	8.65	8.65	8.65
....	....	....	....
3	8.65	8.65	8.65

**Notes:*

- *8.65 m^3/s baseflow was added to each of the hydrographs in order to yield the inflow hydrographs that are reported in* ***Table 5.10****.*
- *The hydrograph was extended up to 3 hours with a constant flow rate of 8.65 m^3/s from 1.2 hours to ensure that the hydrograph was long enough to calculate the peak outflows and maximum water levels at the corresponding times that are beyond 1.2 hours.*

5.8 SEDIMENT

The sedimentation of Impendle and Smithfield Dams are reported in the *Sediment Yield Report (Water Resources Yield Assessment Report: Supporting Document 1)* and sediment deposition in the Smithfield Dam basin is discussed in the *Sediment Deposition and Impact Report (Water Resources Yield Assessment Report: Supporting Document 2)*. Sediment yields of 342 and 317 t/km²/a are recommended for Impendle and Smithfield Dams with 90% confidence.

A first order estimate of the sedimentation for Langa Dam was performed according to the methodology in WRC Report Number 297/2/92, *The Development of the New Sediment Yield Map of Southern Africa*. The first order estimated sediment yield for Langa Dam is 1 165 t/km²/a with 90% confidence.

Based upon this sediment yield, the estimated volume of sediment to accumulate in Langa Dam over 50 years is about 0.21 million m³. The reason for the much higher sediment yield for Langa Dam, in comparison with the sediment yields for Impendle and Smithfield Dams, is Langa Dam's small catchment of 5.34 km² compared to the catchments of Impendle and Smithfield Dams of 1 422 and 2 058 km² respectively.

5.9 RIVER DIVERSION

The following two river diversion phases are proposed for Langa Dam during construction:

- **Phase 1:** A 250 m long cofferdam (Cofferdam 1) that is designed for the recommended peak discharge of 76 m^3/s for the 1:20 year recurrence interval. This cofferdam is required to ensure that river flow remains within the river channel during construction of the two proposed 1.6 m diameter outlet pipes for Langa Dam. The principal data for Cofferdam 1 is summarised in **Table 5.11** below.

- **Phase 2:** A short and low cofferdam (Cofferdam 2) that is designed for the recommended winter peak discharge, for the 1:20 year recurrence interval of 8.20 m^3/s. This cofferdam is required to ensure that river flow is diverted through the two proposed 1.6 m diameter outlet pipes during the construction of the last section of the rockfill embankment for Langa Dam during the winter season.

Table 5.11: Principal data for Cofferdam 1

Parameter	Value
Type of dam	Earthfill
Recommended design flood	1:20 year
Peak discharge for 1:20 year flood	76 m^3/s
Total length	250 m
Crest width	5 m
Side slopes	1.5H:1V
Upstream crest level	890 masl
Downstream crest level	882 masl
Upstream wall height	2 m
Downstream wall height	4 m

5.10 SPILLWAY DESIGN

In terms of the relevant SANCOLD Guidelines the RDF for Langa Dam is the 1:200 year flood. Since Langa Dam has a very small catchment area, and the dam's capacity at the FSL will be about 11 times the MAR, it was initially concluded that Langa Dam will not require a spillway and that excess water above the FSL could be released from the dam via a combined inlet and outlet if needed. This conclusion is based on the fact that the estimated total volume of the SEF is only 0.676 million m^3. The inflow of the SEF into Langa Dam, when the dam is at is FSL, will result into a total volume of 16.346 million m^3 in the dam, which translates to a maximum water level of 923.704 masl in the dam for the SEF, which is only 0.704 m above its FSL of 923 masl.

Although it was concluded that Langa Dam may not require a spillway as per SANCOLD guidelines, it was recognised that Smithfield Dam may continue to supply 8.65 m³/s to Langa Dam during a heavy rainfall event and/or due to a possible operational flaws. Therefore, provision was made for a small spillway to accommodate at least the volume of 8.65 m³/s that is being supplied to the dam. It was also estimated that the maximum flow that can be accommodated by the 1.6 m diameter feeder pipe is 4.64 m³/s when the water level is at FSL of 923 masl. Although the spillway was designed for an overflow of 8.65 m³/s, a smaller ogee spillway can be considered in the detail design for a design overflow of 4.70 m³/s.

The following two spillway options were, however, considered for a design overflow of 8.65 m³/s for the purposes of this feasibility study:

- **Option 1:** A shaft spillway (morning glory) at the lowest point along the dam wall; and
- **Option 2:** An ogee spillway, together with a discharge chute, on the left flank of the dam.

Comparisons of flood and inflow volumes are given in **Table 5.12** and **Table 5.13** respectively.

Table 5.12: Comparison of flood volumes for Langa Dam in terms of the full supply capacity

Full Supply Capacity (million m³)	Recurrence interval (years)								
	2	5	10	20	50	100	200	RMF	SEF
	Estimated flood volumes (million m³)								
14.95	0.043	0.084	0.121	0.164	0.276	0.352	0.441	0.611	0.676
Flood Volumes as percentage of full supply capacity									
-	0.29%	0.56%	0.81%	1.10%	1.85%	2.35%	2.95%	4.09%	4.52%

Table 5.13: Comparison of flood and volumes for Langa Dam in terms of an inflow volume of 8.65 m³/s over 24 hours of 0.747 million m³

Recurrence interval (years)								
2	5	10	20	50	100	200	RMF	SEF
Estimated flood volumes (million m³)								
0.043	0.084	0.121	0.164	0.276	0.352	0.441	0.611	0.676
Flood Volumes as percentage of inflow volume over 24 hours								
5.76%	11.24%	16.20%	21.95%	36.95%	47.12%	59.04%	81.79%	90.49%

5.10.1 Option 1 – shaft spillway

The shaft spillway was designed to provide for free overflow up to the routed peak of the SEF plus 8.65 m³/s base flow. The corresponding shaft spillway has a crest diameter of 4 m and a shaft, as well as conduit diameter of 1.5 m. The total height of the shaft spillway will be 44.05 m and the length of the conduit will be 243 m. The estimated quantities for the shaft spillway option are summarised in **Table 5.14**.

Table 5.14: Summary of estimated quantities for the shaft spillway

Description	Unit	Estimated quantity
Mass concrete	m³	0
Reinforced concrete	m³	740
Hard excavation	m³	536
Concrete pipe (1.5 m diameter)	m	243

The required NOC level for the shaft spillway is 926.6 masl, which is 3.6 m above the FSL. A summary of the flood routing results for the 1:200 year, SEF and 2 times the RMF, together with a base flow of 8.65 m³/s, and the corresponding freeboards for the shaft spillway are given in **Table 5.15**.

Table 5.15: Summary of flood routing results and corresponding freeboards for the shaft spillway

Flood	Base flow	Peak discharge	Total maximum inflow	Maximum outflow	Maximum water level	Available freeboard
	(m³/s)	(m³/s)	(m³/s)	(m³/s)	(masl)	(m)
1:200	8.65	204	212.65	9.48	923.49	3.11
SEF	8.65	313	321.65	16.91	923.72	2.88
2 x RMF	83.65	566	574.65	38.53	924.26	2.34

5.10.2 Option 2 - ogee spillway and chute

The feasibility level design for the ogee spillway was as such to provide for the same NOC level as for the shaft spillway (926.6 masl) in order to compare the two spillway options. The corresponding ogee spillway has a length of 10 m and a 2 m wide by 1 m deep chute with a total length of 280 m. The chute was designed for the routed 1:200 year peak discharge plus 8.65 m³/s. The ogee spillway will only be 1.5 m high and will be founded on rock, of which the level is about 921.5 masl. The stability of this 1.5 m high ogee was also checked for the maximum water level of the routed 1:200 year flood and the estimated safety factor against shear (SF_S) is 26.5, which is much higher than the required minimum SF_S of 4. The estimated quantities for the ogee spillway option are summarised in **Table 5.16**.

Table 5.16: Summary of estimated quantities for the ogee spillway and chute

Description	Unit	Estimated Quantity
Mass Concrete	m³	16
Reinforced Concrete	m³	345
Soft Excavation	m³	2979
Hard Excavation	m³	272

A summary of the flood routing results for the 1:200 year, SEF and 2 times the RMF, together with a base flow of 8.65 m³/s, and the corresponding freeboards for the 10 m long ogee spillway are given in **Table 5.17** for an NOC level of 926.6 m.

Table 5.17: Summary of flood routing results and corresponding freeboards for the ogee spillway

Flood	Base Flow (m³/s)	Peak Discharge (m³/s)	Total Maximum Inflow (m³/s)	Maximum Outflow (m³/s)	Maximum Water Level (masl)	Available Freeboard (m)
1:200	8.65	204	212.65	7.90	923.50	3.10
SEF	8.65	313	321.65	14.44	923.73	2.87
2 x RMF	83.65	566	574.65	35.43	924.27	2.33

5.10.3 Comparison of spillway options 1 and 2

Cost estimates were performed for spillway options 1 and 2, which are the shaft and ogee with chute options respectively. It was found that the shaft spillway will be about 2.3 times more expensive than the ogee spillway and chute. Furthermore, a side spillway is safer than a shaft spillway.

5.10.4 Recommended spillway option for detail design

The ogee spillway with a chute on the left flank is recommended for detail design for the routed 1:200 year peak discharge plus 8.65 m³/s base flow. The preliminary design of the 1.5 m high ogee spillway yielded the following proposed shape for the ogee:

$$y = 0.8442x^{1.85}$$ **(Equation 5-2)**

Where:

y = vertical distance measured from the spillway crest

x = horizontal distance measured from the spillway crest

The discharge table for the proposed 10 m long ogee spillway is given in **Table 5.18**.

Table 5.18: Discharge table for the proposed 10 m long ogee spillway

Water Level (masl)	Flow (m^3/s)
923.00	0.00
923.20	1.73
923.40	5.32
923.60	10.34
923.80	16.61
924.00	23.97
924.20	32.32
924.40	41.58
924.60	51.68
924.80	62.57
925.00	74.19
925.20	86.52
925.40	99.50
925.60	113.12
925.80	127.34
926.00	142.14
926.20	157.51
926.40	173.41
926.60	189.84

5.10.5 Capacity of feeder pipeline to Langa Dam

A maximum flow velocity of 2 m/s will meet the requirement of a lining without damaging the lining. If the permissible maximum flow velocity in the 1.6 m diameter feeder pipeline to Langa Dam is also assumed to be 2 m/s, then the theoretical maximum flow rate that can be achieved in the this pipeline is 4.02 m^3/s. In practice, however, the maximum flow rate of the 2.6 m diameter pipeline, from the tunnel outlet up to where the feeder pipe to Langa Dam branches off, cannot exceed 10.62 m^3/s. If the flow rate in this pipeline exceeds 10.62 m^3/s, then the permissible flow velocity of 2 m/s will also be exceeded. Under the conditions where 10.62 m^3/s is released from Smithfield Dam, and 7.1 m^3/s is supplied to the Baynesfield WTP, then a maximum flow of 3.52 m^3/s can be supplied to Langa Dam. This maximum flow of 3.52 m^3/s to Langa Dam can be achieved when Smithfield Dam spills significantly when the estimated water level is 934.492 masl in Smithfield Dam, which is 4.492 m above Smithfield Dam's FSL of 930 masl and 1.505 m below Smithfield Dam's NOC level of 936 masl.

Should an operational flaw, however, occur and the total release of 8.65 m^3/s from Smithfield Dam is supplied to the 1.6 m diameter feeder pipeline to Langa Dam, then 8.65 m^3/s will flow into Langa Dam until the water level in Langa Dam reaches 905.61 masl, if Smithfield Dam is at its FSL of 930 masl, assuming that Langa Dam was initially empty. When the water level in Langa Dam exceeds 905.61 masl, the flow rate will decrease from Smithfield Dam. If Langa Dam is, however at its FSL of 923 masl, and no water is discharged to the Baynesfield WTP, then the maximum flow rate that can be achieved in the system from Smithfield Dam up to Langa Dam is only 4.64 m^3/s.

A flow rate of 8.65 m^3/s in the 1.6 m diameter feeder pipe will result in a flow velocity of 4.3 m/s, which is in excess of the permissible 2 m/s for the lining, and for this reason the lining of the 1.6 m diameter feeder pipe should be investigated in detail during the detail design.

5.10.6 Time to fill Langa Dam

If no runoff is assumed from Langa Dam's catchment and 1.55 m^3/s, which is the difference between 8.65 m^3/s and 7.1 m^3/s, is supplied to Langa Dam then it will take approximately 117 days to fill up Langa Dam up to its FSL of 923 masl (15.67 million m^3 FSC). If the water level in Smithfield Dam is 934.492 masl and the corresponding 3.52 m^3/s, which is the difference between 10.62 m^3/s and 7.1 m^3/s, is supplied to Langa Dam then it will take approximately 52 days to fill up Langa Dam up to its FSL of 923 masl. The recommended period to fil Langa Dam for planning purposes is, however, 117 days.

Should an operational flaw however occur and the total release of 8.65 m^3/s from Smithfield Dam is supplied to the 1.6 m diameter feeder pipeline to Langa Dam, then 8.65 m^3/s will flow into Langa Dam until the water level in Langa Dam reaches 905.61 masl, if Smithfield Dam is at its FSL of 930 masl. If Langa Dam is initially empty should such an operational flaw occur, then Langa Dam will be filled up to a level 905.61 masl within 5 days (3.81 million m^3) at a flow rate of 8.65 m^3/s. The average flow rate to fill Langa Dam up from a level of 905.61 masl up to its FSL of 923 masl was assumed to be the average of 8.65 m^3/s and 4.64 m^3/s, which equates to 6.65 m^3/s. At the assumed flow rate of 6.65 m^3/s it will take a further 21 days to fill Langa Dam up to its FSL, a further 11.86 million m^3 above a the water level of 905.61 masl. It can therefore be concluded that Langa Dam will be filled up within 26 days, should no water be supplied from Smithfield Dam to the Baynesfield WTP in the case of an operational flaw. During the detail design it should also be investigated how long it will take before Langa Dam overtops if it is at its FSL when an operational flaw occurs and no water is supplied from Smithfield Dam to the Baynesfield WTP.

5.10.7 Freeboard

The required freeboard based on the recommended 10 m long ogee spillway and RDF was determined according to the *SANCOLD Interim Guidelines on Freeboard for Dams*.

These guidelines suggest that the following combinations be considered for a large dam with a high hazard rating:

- **Combination 1:** Sum of the levels for the routed RDF (1:200 year), the wind wave run-up for a 1:25 year event and the wind set-up.
- **Combination 2:** Sum of the levels for the routed RDF (1:200 year), the wind wave run-up for a 1:25 year event, the wind set-up and the flood surges and seiches.
- **Combination 3:** Sum of the levels for the 1:20 year flood, the wind wave run-up for a 1:100 year event, the wind set-up and flood surges and seiches.
- **Combination 4:** Wave height due to an earthquake, alone, was not investigated due to the low seismic horizontal acceleration for the Smithfield Dam site.
- **Combination 5:** Sum of the levels for routed RDF and wave height due to a landslide.
- **Combination 6:** As no flood outlets are foreseen, this combination was not investigated.

The flood surges and seiches are taken as 1 m for large dams.

The results of the above mentioned combinations are summarised in **Table 5.19**.

Table 5.19: Summary of the freeboard determination combinations for recommended spillway

Combination	Routed RDF (height above FSL m)	20-year flood	Wind wave and run-up		Wind set-up	Flood surges and seiches	Earth-quake	Land-slide Wave	TOTAL (m)
			25-year event	100 year event					
Spillway Length = 10 m									
1	0.50	-	1.06	-	0.51	-	-	-	2.07
2	0.50	-	1.06	-	0.51	0.75	-	-	2.82
3	-	0.40	-	1.09	0.51	0.75	-	-	2.75
4	-	-	-	-	-	-	1.13	-	1.13
5	0.50	-	-	-	-	-	-	1.40	1.90

Combination 2 requires the largest freeboard, which is less than the designed freeboard of 3.6 m (NOC 726.60 masl).

A typical section of the proposed spillway is contained in **Annexure 5 D** as **Figure 5.D.1**.

5.11 STABILITY OF DAM EMBANKMENT

Slope stability analyses were conducted with the tested parameters for the different soil types from the geotechnical investigations to determine the optimal slopes of each of the selected dam types. The dam embankment was analysed for steady-state seepage using the grid-and-radius method on the SlopeW software. Parameters used in this exercise are summarised in **Table 5.20**. The results from the soil stability analyses are included in **Annexure 5 B** as **Figure 5.B.1** to **Figure 5.B.12**, with the resultant slopes for the selected dam type summarised in **Table 5.21**.

Table 5.20: Engineering properties for the various material types

Material No.	Material type	Phi – Φ (°)	Cohesion – C (kPa)	Density (kg/m³)
A	Gravel	33	0	1 990
B	Hard rockfill	36	0	2 100
C	Bored rockfill from tunnel	36	0	2 100
D	Completely weathered material from portal excavation	33	8	1 960
E	Dolerite	40	0	2 160
F	Concrete	35	500	2 300

Table 5.21: Resultant slopes for selected dam type

Dam type	Upstream slope	Downstream slope
Rockfill embankment dam	1(V):2(H)	Tested 1(V):2.2(H) and 1(V):2(H)

The slope stability analysis was performed for the rockfill embankment with two different downstream slopes. The downstream slope of 1(V):2.2(H) yielded safety factors high enough to allow the designer to increase the slope. The downstream slope was changed to 1(V):2(H), therefore yielding a more economic design. The estimated safety factors for the critical slip circles for each of the design loading conditions are summarised in **Table 5.22**.

Table 5.22: Summary and comparison of slope stability safety factors for the rockfill embankment

Design loading conditions	Minimum required safety factors		Estimated safety factors for critical slip circle at D/S slope 1(V):2.2(H)		Estimated safety factors for critical slip circle at D/S slope 1(V):2(H)	
	D/S Slope	U/S Slope	D/S Slope	U/S Slope	D/S Slope	U/S Slope
End of construction (no water)	N/A	1.25	N/A	1.6	N/A	1.58
End of construction (no water) and seismic loading	N/A	1.10	N/A	1.6	N/A	1.28
Long-term operational, full reservoir	1.50	1.50	1.72	1.64	1.64	1.64
Rapid draw down	N/A	N/A	N/A	N/A	N/A	N/A
Seismic loading and full reservoir	1.10	1.10	1.36	1.17	1.31	1.18
Seismic loading and rapid draw down	N/A	N/A	N/A	N/A	N/A	N/A

5.12 DAM OUTLET AND INLET TOWER DESIGN

5.12.1 General arrangement

From the limnological study by Umgeni Water, *Water Quality and Limnological Review (P WMA 11/U10/00/3312/3/1 – Write-up 2)*, the following conditions were described to exist at Langa Dam:

With a maximum water depth of 38 m and long retention times, the proposed dam will also display thermal and chemical stratification during the summer period. This is particularly likely due to its sheltered location, low inflows and likely low wind mixing. However, a variable abstraction/environmental release mechanism in Langa Dam is not recommended because:

- Langa Dam is planned to be used in intervals of approximately 10 years for a period of 3 weeks during tunnel maintenance;
- The preferred time of year to undertake tunnel maintenance would be during the winter isothermal period;
- During the 3 week duration of tunnel maintenance, the majority of the water in Langa Dam will be used for water treatment, making a variable abstraction mechanism redundant; and
- Even if, under emergency conditions, the water from Langa Dam is required to be abstracted for treatment during the summer stratified period, the activated carbon dosing facility planned for the water works should be able to treat the water to potable water standards.

In terms of environmental releases from Langa Dam:

- Since the Langa Dam is located in a very small, upper uMlaza catchment, once the dam has been filled, it is likely to remain full until used. Under these conditions, the natural spilling from the dam is likely to satisfy the environmental releases from a quality and quantity perspective. However, a dam scour/river release mechanism will be required to supplement environmental releases when necessary and ensure that adequate water of acceptable quality for environmental flows is released at all times.
- Consideration will have to be given to environmental releases (approximately 0.4 m^3/s) immediately after the tunnel maintenance period since the filling time for this dam is long (3.9 years) and thus would take a long time to refill, during which time only scour releases would be possible if Langa Dam was not supplemented from the proposed Smithfield Dam.

A multi-draw-off system is therefore not required; however, a dual pipe system for maintenance reasons is provided.

The layout will consist of the following:

- A tower outlet/inlet structure is proposed, to serve the following purposes:
    - An inlet structure for water that will be supplied to Langa Dam from Smithfield Dam;
    - An outlet structure for water that will be supplied to Baynesfield WTP from Langa Dam; and
    - An outlet structure for environmental releases from Langa Dam.

- A cylindrical tower with an inside diameter of 7.2 m is proposed for the outlet tower in order to accommodate the following two 1.6 m diameter pipes at the bottom. These will be for water that is supplied to Langa Dam from Smithfield Dam, and for water that is supplied from Langa Dam to Baynesfield WTP. One 1.6 m diameter outlet pipe will suffice for water supply to the Baynesfield WTP, but for maintenance reasons DWA requires a dual outlet pipe system. While refurbishing one system the other system must be in working condition for immediate supply if required. It is proposed that environmental releases into the downstream water course be released through one of the 1.6 m diameter pipes.

The proposed configurations of the outlet tower that should be considered for detail design are the following:

- A cylindrical structure with an outside diameter of 11.2 m and an inside diameter of 7.2 m; or
- A square tower of 8.2 m x 8.2 m with a cylindrical shaft with a diameter of 7.2 m.

The proposed two 1.6 m diameter outlet pipes will be able to draw down Langa Dam from its FSL to 50% within 9.6 days, and the draw down to the lowest level will be achieved in less than 60 days. These drawdown periods comply with DWA's draw down requirements of 60 days, or less, for draw down to 50% of the dam height and 120 days, or less, for draw down to 10% of the dam height.

5.12.2 Down stream protection layer of rockfill embankment

From an environmental point of view the proposed dolerite downstream protection layer for the embankment may not be acceptable, and provision may need to be made for vegetation of the downstream slope of the embankment. If found that the dolerite downstream protection layer is not acceptable, the downstream slope layers may be replaced with the following layers:

- A 600 mm thick gravel transition layer on top of the completely weathered material from the tunnel excavation (spoil);
- A finer 600 mm thick finer gravel layer, close to sand, on top of the afore-mentioned gravel layer, and
- A 150 mm topsoil layer on top of the afore-mentioned finer gravel layer that will be vegetated.

A schematic of these proposed layers on the downstream slope, for both the dolerite and vegetation options, is shown in **Figures 5.A.2** and **5.A.3**, see **Annexure 5 A** in **Volume 2** of this report.

The required volumes of the transition and topsoil layers on the downstream slope are 21 700 m³, 21 700 m³ and 5 450 m³, respectively. This option for the downstream slope will cost about R 11.23 million more in terms of capital costs than the initial proposed dolerite downstream slope protection. This cost difference would be larger if the entire life-cycle cost is considered, due to the higher operational and maintenance costs involved with the proposed alternative downstream protection layer.

The cost comparison for the downstream slope protection, in terms of both options, is given in **Table 5.23**. This option for downstream slope protection will however need to be re-assessed during the final design of Langa Dam.

Table 5.23: Cost comparison for Langa Dam's downstream slope

Proposed vegetation alternative for the downstream slope				
Description	**Unit**	**Quantity**	**Rate**	**Amount (excl. VAT)**
Gravel transition layer (600 mm thick)	m³	21 700	R 210	R 4 557 000
Finer transition layer (600 mm thick)	m³	21 700	R 670	R 14 539 000
Topsoil layer (150 mm thick)	m³	5 450	R 102	R 555 900
Additional hard rockfill material on top portion to replace dolerite	m³	36 500	R 120	R 4 380 000
Vegetation (hydro-seeding)	Ha	4	R 7 205	R 28 820
Estimated total additional cost for the proposed alternative	-	-	-	**R 24 060 720**
Proposed dolerite alternative for the downstream slope				
Original estimated cost for the proposed dolerite layer (4 m thick)	m³	144 620	R 310	R 12 830 900
Original estimated cost for the proposed dolerite layer (4 m thick)	-	-	-	**R 12 830 900**
Estimated additional cost for downstream slope protection (Excl. VAT)	-	-	-	**R 11 229 820**

Figure 5.1 and **Figure** 5.2 are artistic impressions of Langa Dam during the operational phase for both the vegetation and dolerite options, respectively.

Figure 5.1: Artistic impression of Langa Dam with vegetation downstream protection layer

Figure 5.2: Artistic impression of Langa Dam with dolerite downstream protection layer

5.13 PRINCIPAL DATA FOR LANGA DAM

The principal data for Langa Dam is summarised in **Table 5.24**.

Table 5.24: Principal data for Langa Dam

Parameter	Description
Type of dam	Concrete Faced Rockfill Dam (CFRD)
Catchment area	5.34 km^2
Recommended design flood (RDF)	1:200 year
Peak inflow of the 1:200 year flood	204 m^3/s
Regional maximum flood (RMF)	283 m^3/s
Safety evaluation flood (SEF)	313 m^3/s
Full supply level (FSL)	923.00 masl
Minimum operating level (MOL)	898.24 masl
Non overspill crest level (NOC)	926.60 masl
Gross storage volume at FSL, including additional storage created by the quarry	15.67 million m^3
Live storage volume at FSL, including additional storage created by the quarry	14.82 million m^3
Area at full supply level	95.48 ha
Estimated sediment volume after 50 years	0.21 million m^3
Mean annual runoff (MAR)	2.03 million m^3 per annum
Maximum wall height of the embankment	46.60 m
Maximum wall width of the embankment	202.72 m
Time of supply at 7.10 m^3/s	24 days

5.14 ACCESS DESIGN

The proposed access road to Langa Dam is along an existing gravel road from the proposed position of the tunnel outlet portal towards the plantations east of Langa Dam. The existing gravel road will have to be upgraded for just over 1 km up to Langa Dam in order to accommodate the construction traffic. It is proposed that the existing gravel road be upgraded to an 8 m wide layered and compacted gravel road.

5.15 COST ESTIMATE

A detailed cost estimate of all construction activities of Langa Dam, comprising quantities and rates, has been completed, and is contained in **Annexure 5 C**. **Table 5.25** shows a summary of this cost estimate. Assumptions made in determining all cost estimates are described in **Section 14**. The total scheme cost estimate with all components added together is given in **Section 14.4**.

Table 5.25: Summary of cost estimate of activities for Langa Dam

Description	Cost (R million, excl. VAT)
River diversion works	1.4
Development of quarry	0.5
Langa Dam main embankment (concrete face rockfill dam)	315.8
Spillway	3.6
Outlet pipes	12.8
Outlet works, intake structure	47.1
Miscellaneous	57.7
TOTAL	**438.8**

5.16 RECOMMENDATIONS

For Langa Dam it is recommended that:

- Further geotechnical investigations are required during tender design regarding the bottom outlet;
- The spillway be designed for 4.70 m³/s in the detail design, which could be a much simpler spillway, instead of 8.65 m³/s; The lining for the 1.6 m diameter feeder pipe to Langa Dam be further investigated during the detail design, since velocities in excess of 4.3 m/s could occur in this pipeline due to operational flaws;
- Operational flaws are investigated in detail during the detail design for the case where no water is supplied from Smithfield Dam to the Baynesfield WTP due to an operational flaw, while Langa Dam is already at its FSL, and
- The vegetation option for downstream slope protection be re-assessed during the final design.

5.17 REFERENCES

AECOM, AGES, MMA & Urban-Econ, 2014. *The uMkhomazi Water Project Phase 1: Module 1: Technical Feasibility Study: Raw Water; P WMA 11/U10/00/3312/3/1/5 - Supporting document 5: Dam type selection*, Pretoria, South Africa: Department of Water Affairs (DWA).

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Water Research Commission, 1992. *The Development of the New Sediment Yield Map of Southern Africa. WRC Report No 297/2/92*. Compiled by Sigma Beta Consulting Engineers.

Water Research Commission, 2011. *Guidelines on freeboard for dams, Volume II. WRC Report No 1759/2/11*. South African Water Research Commission.

6 BAYNESFIELD RAW WATER PIPELINE

The layout of the entire scheme is shown on **Figure 1.2** in **Section 1.2** of this report. The routes for the raw water pipelines to the WTW and Langa Dam are shown in more detail on **Figure 6.1**.

These raw water pipelines will supply raw water from Smithfield Dam to the WTW as well as to Langa Dam for storage whenever raw water needs to be supplied from Langa Dam to the WTW via the same pipelines. Various positions of water treatment plants have been considered in the Module 3 study. The treatment plant as shown in **Figure 6.1** has been selected for the layout of the pipeline.

Figure 6.1: Pipeline route

6.1 DESIGN LAYOUT PHILOSOPHY

The end of the tunnel from Smithfield Dam will be located about 1 500 m downstream of Langa Dam and the raw water pipeline to the WTW will start at this point. The convergence of the 3.5 m diameter tunnel to the 2.6 m diameter steel pipeline is discussed in detail in **Section 4.6.1**. The 2.6 m diameter raw water pipeline will convey raw water from the end of the tunnel to the WTW at a peak flow rate of 8.65 m³/s. A take-off along the raw water pipeline is proposed, as shown in **Figure 6.1**, to convey raw water to Langa Dam for storage. This take-off will be located approximately 1 500 m downstream of Langa Dam and the diameter of this take-off pipeline will be 1.6 m. The total length of the raw water pipeline from the end of the tunnel up to the WTW is about 5.2 km. The design layout philosophy is as follows for the following two supply scenarios:

Scenario 1: Direct supply from Smithfield Dam to the WTW

- Raw water will be released from Smithfield Dam into the proposed 3.5 m diameter tunnel at a flow rate of 8.65 m^3/s.
- At the end of the tunnel, raw water will be discharged into the proposed 2.6 m diameter gravity steel pipeline to supply the WTW, also at a flow rate of 8.65 m^3/s.
- During off-peak periods, and when Smithfield Dam reservoir is at high levels, the pipeline will be closed and raw water will be diverted to the take-off from where it will be conveyed to Langa Dam for storage (see **Section 5.2** of this report for details of the operation rule for Langa Dam).

Scenario 2: Supply from Langa Dam to the WTW

- During maintenance periods of the tunnel, when raw water cannot be conveyed from Smithfield Dam via the tunnel, the stored water in Langa dam will then be supplied via the pipeline under gravity to the WTW, at a flow rate of 8.65 m^3/s, for the duration of the maintenance of the tunnel.

6.2 HYDRAULIC DESIGN

The hydraulic design of the tunnel and pipeline is reported in **Section 4.3** of this report, except for surge that is discussed further on in **Section 6.4.1**. The hydraulic design of the 2.6 m diameter pipeline is described in **Section 4.3**. The water supply velocity in the pipeline is 1.63 m/s, which is less than 2 m/s. This meets the requirement of a lining without damages.

6.3 FOUNDATION AND CONSTRUCTION MATERIALS INVESTIGATION

The areas to be traversed by the proposed water pipeline are mainly underlain by firm to stiff silty clay or clayey silt containing sand, gravel, cobbles or boulders. In-situ material excavated during trenching will be suitable for use both as selected layer(s) in pavement and as backfill, and marginally suitable as bedding material in water pipeline construction. The western section of the water pipeline traverses a stream and recognisable wetland, therefore unstable sidewall conditions are envisaged during trenching.

To ensure acceptably stable sidewalls in areas having a potential for unstable sidewalls during pipeline construction, trench excavations should not advance too far ahead of water pipeline placement i.e. water pipeline placement and subsequent backfilling should proceed immediately after excavation. The risk of unstable sidewalls may be mitigated by benching or battering of trenches so as to maintain adequate levels of safety.

In-situ material excavated during trenching will be suitable for use both as selected layer(s) in pavement and as backfill, and marginally suitable as bedding material in water pipeline construction.

6.4 CONCEPTUAL DESIGN

The conceptual design of the 2.6 m diameter steel pipeline covers the following key aspects:

- Basic surge analysis;
- Pipe wall thickness;
- Positioning of scour valves and manholes criteria;
- Positioning of thrust blocks;
- Backfill, bedding and excavation; and
- Stilling basin at the WTW.

These key aspects are discussed in more detail in the subsequent sub-sections of **Section 6.4** below.

6.4.1 Basic surge analysis

Basic surge calculations were performed for the proposed 5 120 m long raw water steel pipeline from the tunnel to the WTW. The estimated period of the pipeline, which is the time that it will take the surge wave to move up and down the pipeline, was calculated as 9.85 s. This means that the Joukowsky equation to calculate maximum change in pressure will be valid for any event that takes less than 9.85 s to cause a surge, e.g. a sudden valve closure or pump trip.

The pipeline period of 9.85 seconds was calculated with the following equation:

$$T = \frac{2L}{C}$$ **(Equation 6-1)**

Where:

T	=	Pipeline period (seconds)
L	=	Pipeline length (m)
C	=	Wave celerity (m/s), which is approximately 1000 m/s for steel

The maximum change in pressure of 1.63 MPa was calculated with the Joukowsky equation, which is as follows:

$$\Delta H = \frac{C.\Delta V}{g}$$ **(Equation 6-2)**

Where:

ΔH	=	Maximum change in pressure (m)
ΔV	=	Change in flow (m/s) velocity due to the event causing the surge
g	=	9.81 m^2/s

$$\Delta P = \rho.g.\Delta H.10^{-6}$$ **(Equation 6-3)**

Where:

ΔP	=	Maximum change in pressure (MPa)
ρ	=	1000 kg/m^3 for water

The maximum and minimum pressures along the 5 120 m pipeline, due to the maximum change in pressure as a result of surge, were estimated and are as follows:

- Maximum pressure due to surge = 1.86 MPa; and
- Minimum pressure due to surge = -1.52 MPa.

The assessment of surge for the purposes of the feasibility design was a very basic assessment, and therefore much more thorough assessments of surge are recommended during the detail design of the pipelines. It should be noted that the hydropower house operation will also cause higher pressures that should be taken into account. Preliminary indications show that these maximum pressures at the power house are approximately 50% of the maximum static pressure at Smithfield Dam.

It is also recommended that the hydropower plant’s surge (water hammer) effects on conveyance be considered in the final design.

6.4.2 Pipe wall thickness

A basic calculation was performed, in order to estimate the minimum required pipe wall thickness of about 22.5 mm, with the following equation:

$$C = \sqrt{\frac{1}{\rho.\left\{\frac{1}{K} + \frac{C_1 . D}{T . E}\right\}}} \qquad \textbf{(Equation 6-4)}$$

Where:

C = Wave celerity (m/s), which is approximately 1000 m/s for steel

ρ = 1000 kg/m³ for water

K = Bulk modulus for water (2.1 x 10^9 N/m²)

D = Pipe diameter (m)

T = Pipe wall thickness (m)

E = Bulk modulus for steel (2.1 x 10^{11} N/m²)

$C_1 = {}^5/_4 - \eta$, With η = Poisson’s Ratio (0.3 for steel)

The economical thickness of the pipe can be determined by using a slenderness ratio (D/t) of approximately 145. According to this calculation, the economical thickness of the 2.6 m diameter pipe should be roughly 18 mm. However, due to the results of the water hammer analysis and to account for the potential of hydropower, the greater thickness of 22.5 mm should be adopted, with further investigations into pipe strengthening during the detailed design phase.

6.4.3 Positioning of scours, valves and manholes criteria

The positioning of air valves and scours depends on the longitudinal section of the pipeline. Air valves will be required at all the high points and scours will be required at all the low points along the pipeline. A total of 20 high points, where air valves will be required, and a total of 27 low points, where scours will be required, were identified along the proposed route of the 5 120 m long pipeline from the tunnel to the WTW. A minimum of 20 manholes, to accommodate the air valves, and 27 scour chambers will be required along the 5 120 m pipeline. The number and types of valves as well as scours along the pipeline are not limited to the numbers reported in this report, but will depend upon the final design and surge analysis for the pipeline.

The operation rule for emptying the conveyance system will be proposed as follows:

- Close the valve located downstream of the tunnel outlet.
- Empty the pipeline by supplying water to the treatment plant.
- Drain the incremental part of the pipe by its scour
- Limit the discharge into the stream as per EIA requirements (the detail design must include the limitations)

6.4.4 Positioning of thrust blocks

Since a continuous weld pipeline is proposed, no thrust blocks will be required; this must however be confirmed in the final design.

6.4.5 Backfill, bedding and excavation

The backfilling and bedding should preferably comply with the following sections of the DWA Specification DWS 1110, Construction of Pipelines:

- Section 3.16 Backfill Material;
- Section 7.1 Excavations; and
- Section 7.2 Backfilling.

The desired properties of the bedding materials, as specified in DWA Specification DWS 1110, are summarised in **Table 6.1** below.

Table 6.1: Desired material properties for bedding materials

Type of Bedding	Material	Percentage by Mass Passing Screens				Maximum Atterberg Limits		
		9.5 mm	4.75 mm	0.425 mm	0.002 mm	LL (%)	PI (%)	LS (%)
Type A	Finely Graded	100	100	80-100	0-45	30	15	5
Type B	Medium Graded	100	80-100	60-80	0-40	35	18	7.5
Type C	Granular	100	70-100	30-60	0-35	40	20	10

In terms of the abovementioned specification, the trench width should not be less than 4.2 m for the proposed 2.6 m diameter pipe. The depth of the trench will be in accordance with the approved drawings and should be controlled as such to ensure that a uniform depth of bedding underneath the pipeline is ensured. The minimum desired depth of the trench is 1.5 m.

6.4.6 Stilling basin at the WTW

The outflow from the hydropower plant (HPP), if implemented, will have to be discharged into a stilling basin in order to obtain the desired inflow velocity of about 1.5 m/s into the WTW. A preliminary design was performed for this stilling basin and a drawing thereof is included in **Annexure 6 A** as **Figure 6.A.3**. For the purposes of this study a 3.5 m long hydraulic jump type stilling basin, of which the width varies from 2.6 to 3.5 m, was designed. It is, however, recommended that USBR Type II and III stilling basins also be investigated during the detail design stage.

6.5 DRAWINGS

The following preliminary drawings are attached in **Annexure 6 A** of this report:

- Raw water pipeline layouts;
- Raw water pipeline longitudinal sections; and
- Stilling basin details at Baynesfield WTP.

Typical general details have not been included in this feasibility level study, as they are standard DWA drawings.

6.6 COST ESTIMATES

A detailed cost estimate of all construction activities of the Baynesfield Raw Water Pipeline, comprising quantities and rates, has been completed, and is contained in **Annexure 6 B**. **Table 6.2** shows a summary of this cost estimate. Assumptions made in determining all cost estimates are described in **Section 14**. The total scheme cost estimate with all components added together is given in **Section 14.4**.

Table 6.2: Summary of length and cost estimate of activities for the Baynesfield Raw Water Pipeline

Description	Length (m)	Cost (R million, excl. VAT)
Pipeline – 2.6 m diameter section	5 120	277.3
Pipeline – 1.6 m diameter section	1 250	27.0
Miscellaneous (establishment of a sub-consultant)	-	60.9
TOTAL	**6 370**	**365.2**

6.7 RECOMMENDATIONS

The layout of the pipeline should be confirmed after the water treatment plant location has been fixed.

The discharge during scouring of the raw water pipeline should be minimised, to be confirmed during detail design.

7 HYDROPOWER PLANTS

As part of the uMWP Technical Feasibility Design, an assessment of the feasibility of hydropower generation as a secondary benefit to the uMWP was undertaken. This assessment is described in detail in the *Hydropower Assessment Report (P WMA 11/U10/00/3312/3/3)*, and a summary of the main findings are highlighted below.

7.1 POSSIBLE SITES

Two potential sites were identified; the first being at the Baynesfield WTW as part of the conveyance structure from Smithfield Dam to Baynesfield WTW, and the second just below Smithfield Dam on the outlet works. At the first site, known as Baynesfield Hydropower Plant (HPP), power would be generated by water transfers through the conveyance structure. At the second site, known as Smithfield Dam Hydropower Plant, power would be generated by spills and releases from the dam.

7.2 ENERGY YIELD

The Water Resource Planning Model (WRPM) was used to simulate the future dam levels and flow volumes over the project period, which were used to determine the hydropower potential at each site for key probabilities.

Figure 7.1 shows the time series of hydropower potential over the project period for Baynesfield HPP. **Figure 7.2** shows the probability distribution curve of hydropower potential for Smithfield Dam HPP.

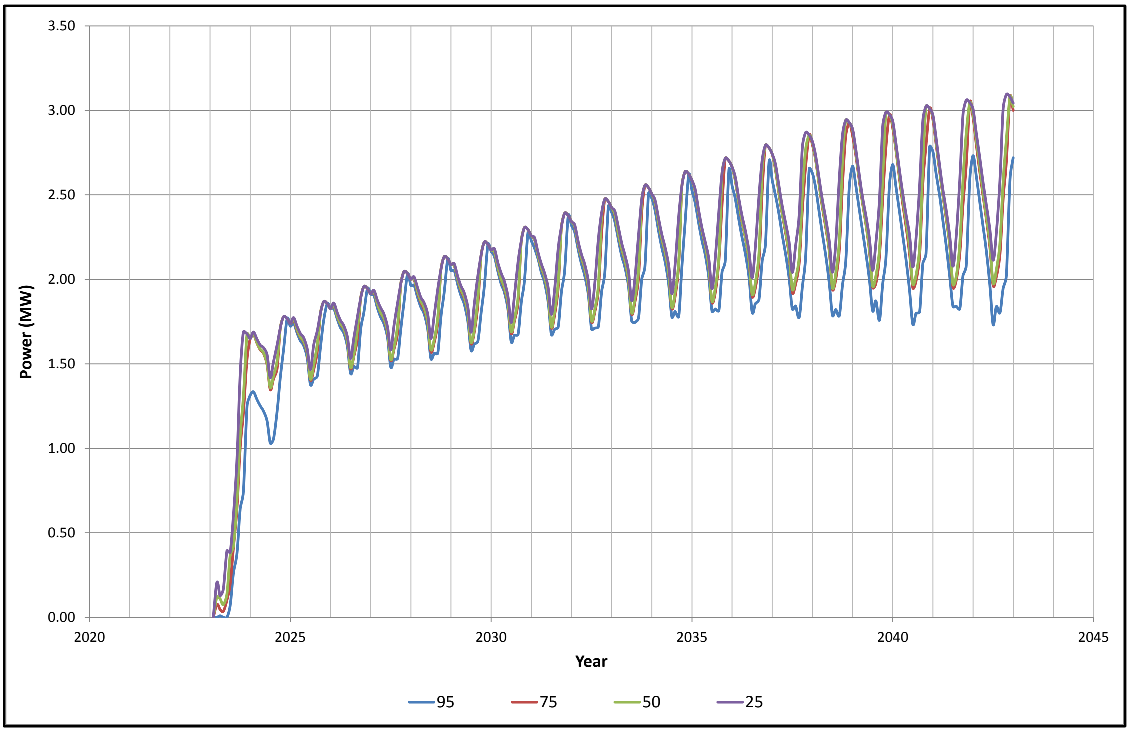

Figure 7.1: Hydropower potential at Baynesfield HPP

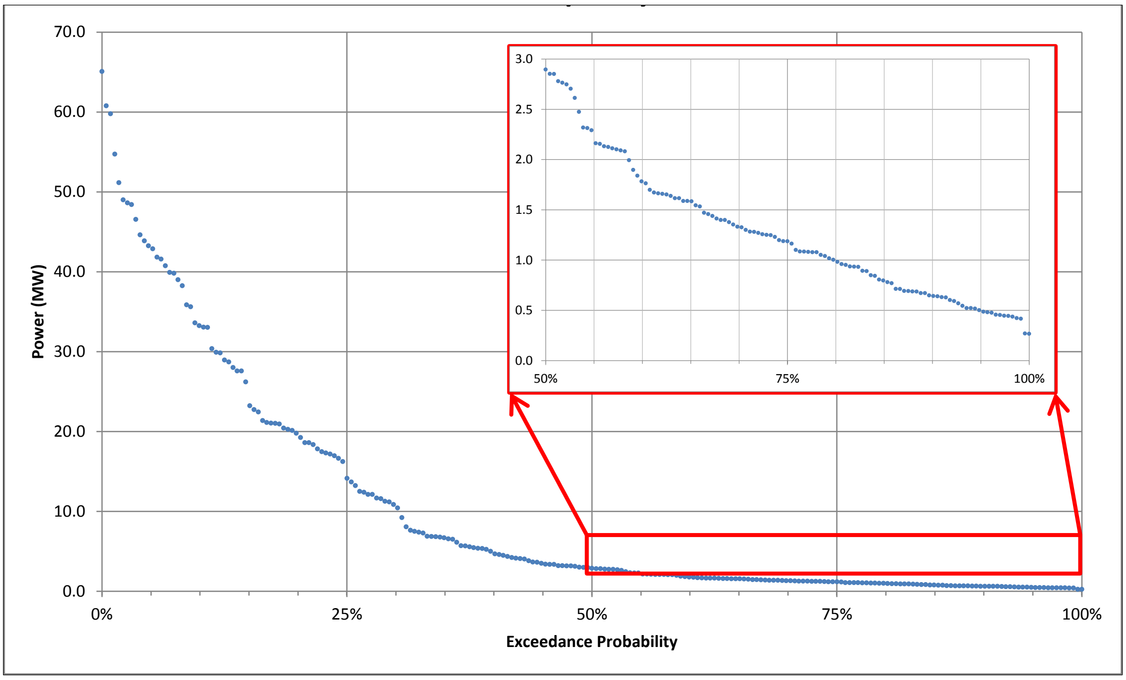

Figure 7.2: Hydropower potential at Smithfield Dam HPP

7.3 CONCEPTUAL DESIGN OF HYDROPOWER PLANTS

Having calculated the hydropower potential at each of the sites, the conceptual design of the HPPs was done. This entailed the design of turbines, including design for water hammer effects; the layout of the HPPs; and the design of the power transmission.

7.3.1 Baynesfield HPP

At Baynesfield HPP, the rated point was calculated as 8.65 m^3/s flow and 41.7 m net head, with 3 MW power potential. In order to accommodate the effects of water hammer, and because of the long penstock leading up to the turbine, a bypass to the turbine would be needed, in addition to a slow closure of the turbine. A flywheel would also be needed to limit the speed rise.

The design of power transmission infrastructure is dependent on the usage of the power. Due to the existence of infrastructure for providing power to the site for operation of Baynesfield WTW, as well as during its construction, the main additional requirement would be nominal infrastructure to “clean” the generated power for wheeling into the grid or for direct use by the WTW. In addition, short underground or overhead cables would be needed, for about 50 m.

The layout of the powerhouse is given in **Annexure 7 A**.

The alternatives for the HPP at this site were as follows:

- Baynesfield HPP alternative 1: Power wheeled into national grid for use at Baynesfield WTW; and
- Baynesfield HPP alternative 2: Power supplied directly into Baynesfield WTW with supply from the national grid as backup.

7.3.2 Smithfield Dam HPP

At Smithfield Dam HPP, two power generation alternatives were considered, with turbines rated 0.5 and 2.6 MW. The rated point for 0.5 MW was calculated as 1.1 m^3/s flow and 55.5 m net head; and for 2.6 MW was 5.0 m^3/s flow and 64.0 m net head. Because of the short penstock length, no bypass pipe would be needed to limit water hammer, and the turbine could have a short closure time with acceptable pressure rise. A flywheel would be required to limit speed rise.

Modifications will need to be made to the dam’s outlet works in order to accommodate the potential powerhouse. This would involve the following:

- A bypass pipe to accommodate the turbine, which will allow the turbine to not interfere with the operation of the outlet works during emergency releases;

- A connection between the two pipes, so that hydropower can be generated when maintenance is done on either of the pipes;
- Five additional butterfly valves, in order to control flow in the abovementioned connection and to the turbine; and
- Larger sleeve valves to accommodate additional losses incurred.

The layout of the potential powerhouse incorporated into the dam's outlet works is illustrated in **Figure 7.3**.

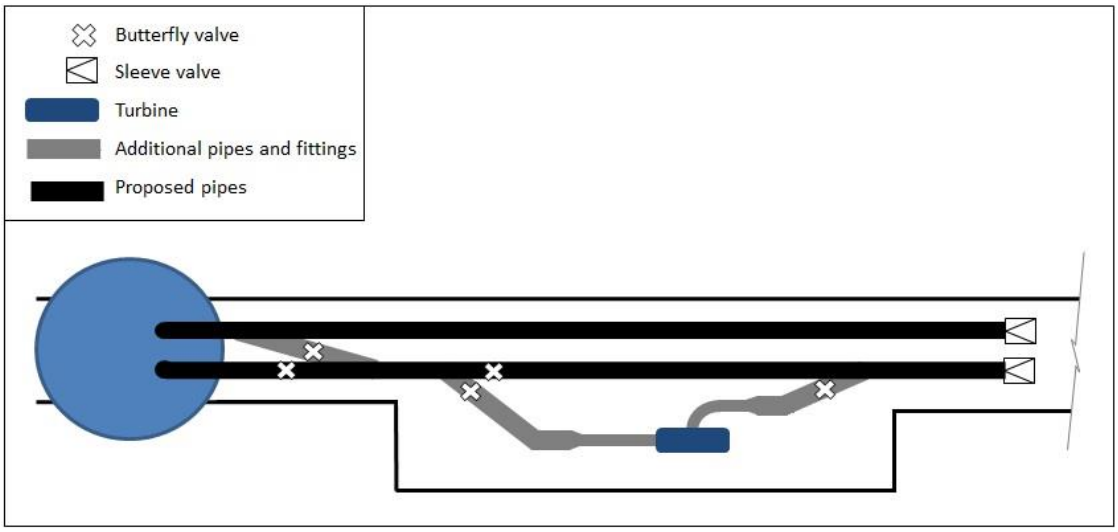

Figure 7.3: Schematic of modifications to outlet works for Smithfield Dam HPP

If the powerhouse be considered feasible, the details of these modifications must be confirmed during detail design as it will also depend on the selected turbine configuration. The superstructure of the powerhouse would be similar to that for Baynesfield HPP. Power transmission infrastructure will be the same as with Baynesfield HPP, with 500 m of transmission lines.

Preliminary cost estimates have been made, incorporating the abovementioned potential modifications, and are included in **Section 7.4**.

The alternatives for the HPP at this site are as follows:

- Smithfield Dam HPP alternative 1: Power wheeled into national grid for operation and maintenance of Smithfield Dam (0.5 MW turbine);
- Smithfield Dam HPP alternative 2: Power wheeled into national grid for operation and maintenance of Smithfield Dam (2.6 MW turbine); and
- Smithfield Dam HPP alternative 3: Power supplied directly to Smithfield Dam operation and maintenance facilities.

7.4 CAPITAL AND OPERATION AND MAINTENANCE COST ESTIMATES FOR DETERMINING SUSTAINABILITY

The capital and operation and maintenance (O&M) costs are summarised in **Table 7.1** below. Exact power transmission costs for Baynesfield HPP alternative 2 are not known at this stage, but the costs for alternative 1 would be indicative of these costs. Also, the civil, hydro-mechanical and power transmission costs of Smithfield Dam HPP alternative 3 are presently not known. Should these alternatives be pursued further, detailed investigations into the exact infrastructure requirements and the costs thereof must be done. For this reason, the two abovementioned alternatives will not be considered in detail further, but will still be remarked on.

Table 7.1: Capital and O&M cost estimates

HPP alternative	Capital cost (R'000)				Annual O&M cost (R'000)
	Civil works	Hydro-mechanical	Transmission line	Total	
Baynesfield HPP alternative 1	3 748	36 968	2 075	**42 791**	**1 571**
Smithfield Dam HPP alternative 1	2 542	12 647	2 750	**17 939**	**622**
Smithfield Dam HPP alternative 2	3 748	30 082	2 750	**36 580**	**1 323**

7.5 EVALUATION OF ECONOMIC SUSTAINABILITY

For direct consumption of the power, the potential revenue would be equal to the money not spent on buying power from Eskom. For wheeling power into the grid, the revenue would be similar to this, but charges would be applicable by the generator for the delivery of the energy.

Based on the costs and potential revenue associated with the HPP alternatives, net present values (NPVs) were determined for the life-cycle of the project, and are shown in **Table 7.2** below.

Table 7.2: NPVs for HPP alternatives

HPP alternative	Net overall benefit at certain discount rate (R'000)		
	6%	8%	10%
Baynesfield HPP alternative 1	22 605	10 366	3 666
Smithfield Dam HPP alternative 1	443	-1 213	-1 970
Smithfield Dam HPP alternative 2	31 896	18 553	10 638

The above table shows that wheeling into the grid is feasible at both sites; however, for Smithfield Dam HPP, it will only be feasible for higher hydropower generation. One point should be noted:

- HPPs can be implemented at both the Bayensfield and Smithfield Dam sites, as the water which generates the power is independent for each site;

7.6 Conclusions and Recommendations

Based on the assessment of the economic feasibility of the HPP alternatives, it was found that wheeling power into the grid is a feasible option for both Baynesfield HPP and Smithfield Dam HPP. For the latter, high hydropower generation is needed for sustainability. These options should be discussed with Umgeni Water to determine whether they would be interested in such an arrangement, and should also be discussed with Eskom regarding the selling price.

Two options requiring further investigation into the infrastructure requirements and costs are: direct supply of power into Baynesfield WTW, and direct supply of power to the operation and maintenance of Smithfield Dam. If they are found to be feasible, they should also be discussed with and approved by Eskom.

Further investigations should also be done to identify parties that would be interested in linking the scheme to a renewable energy program for small hydropower schemes. This arrangement should also be discussed with and approved by Eskom.

7.7 Cost Estimates

A detailed cost estimate of all construction activities for both of the recommended HPPs, comprising quantities and rates, has been completed, and is contained in **Annexure 7 B**. **Table 7.3** shows a summary of this cost estimate. Assumptions made in determining all cost estimates are described in **Section 14**. The total scheme cost estimate with all components added together is given in **Section 14.4**.

Table 7.3: Summary of cost estimate of activities for both hydropower plants

Description	Cost (R million, excl. VAT)
Baynesfield HPP (alternative 1)	42.8
Smithfield Dam HPP (alternative 2)	36.6
Miscellaneous	4.0
TOTAL	**83.3**

7.8 REFERENCES

AECOM, AGES, MMA & Urban-Econ, 2014. *The uMkhomazi Water Project Phase 1: Module 1: Technical Feasibility Study: Raw Water; P WMA 11/U10/00/3312/3/3 - Hydropower assessment report*, Pretoria, South Africa: Department of Water Affairs (DWA).

8 FLOW GAUGING WEIRS

8.1 BACKGROUND

Three gauging weirs are required to monitor flows in the uMkhomazi River. The following positions and purposes are described below. The locations are also shown on **Figure 8.1**. They are as follows:

- Weir 1: Upstream of Smithfield Dam to measure inflow to Smithfield Dam.
- Weir 2: Downstream of Smithfield Dam to determine the lower portion of discharges from Smithfield Dam and to monitor in-stream flow requirements.
- Weir 3: Near EWR/IFR2, further downstream of Smithfield Dam. This will determine the runoff from the incremental catchment downstream of Smithfield Dam to assist with determining and monitoring the ecological water requirement.

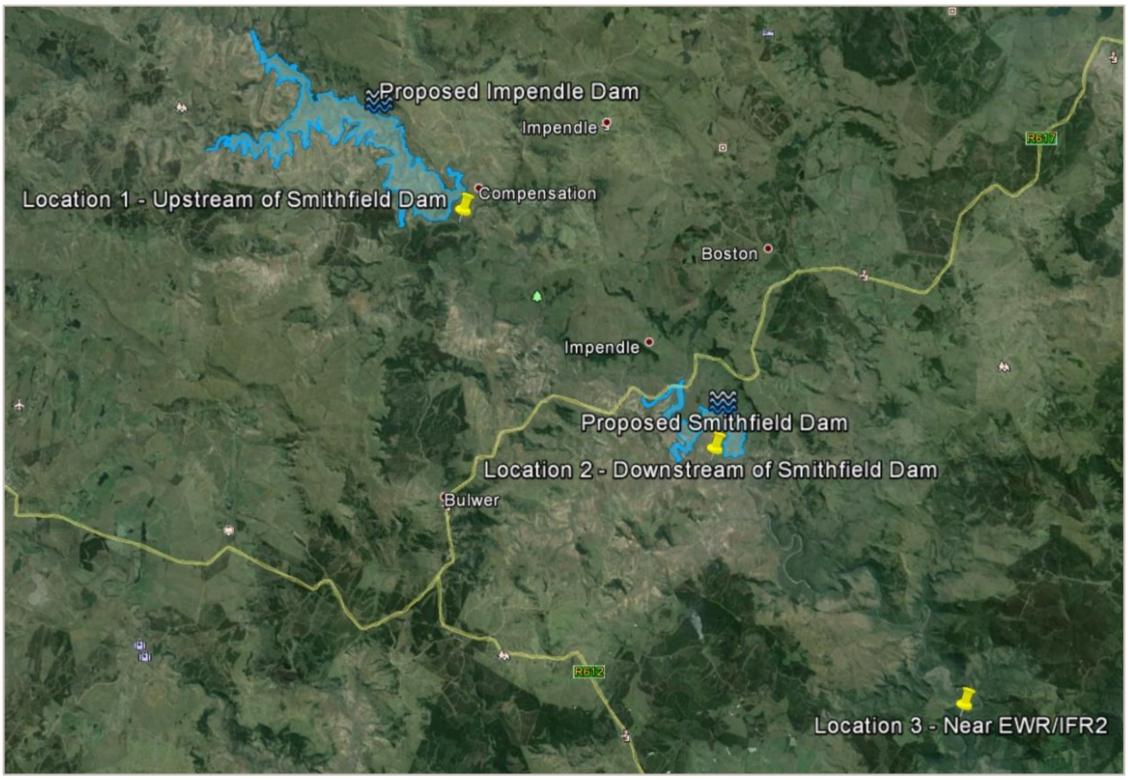

Figure 8.1: Locations of the proposed three gauging weir sites

This chapter describes the feasibility design of these gauging weirs, and the associated cost estimates for inclusion in the total cost estimate for the project.

8.2 DESIGN REQUIREMENTS

8.2.1 Hydrographical survey

Three cross sections through the uMkhomazi River were executed at the three chosen gauging weir locations. The average total river length surveyed was approximately 57 m and the width across the river surveyed approximately 286 m.

8.2.2 Backwater analysis

The topographical survey was reduced and five cross sections were interpolated and exported to the HEC-RAS software. This software produced information used in the determination of the stage versus flow graph, water cross sectional area and top width at all five interpolated cross sections. This information was required for the design of the gauging weirs. Graphs depicting the upstream area and top widths as well as the downstream water level versus flow graphs are shown in the relevant sections.

8.2.3 Flow measurement and sizing requirements

The design of a flow-gauging weir is dependent on the size of flow that needs to be measured accurately. The DWA suggests that ideally the maximum flow needed to be measured accurately is 95% of the expected runoff volume in the river. However, very few gauging weirs in South Africa adhere to this requirement as it results in very large gauging weirs. For the purposes of this feasibility study, 75% of the expected runoff volume was used for the design flow of the gauging structures, where relevant.

The minimum flow requirement is determined by the following:

- The length of the first notch
- The requirement that a minimum flow of 50 mm over the crump weir is necessary for accurate flow measurement
- The practical length of the notch in relation to the river cross section and height

A first notch width of 15 m can therefore accurately measure all flow above 0.33 m^3/s.

8.2.4 Weir design

The gauging weir was designed according to the report TR126 (Van Heerden, Van der Spuy, & Le Roux, 1986). The flow capacity of the horizontal crump weir is determined with **(Equation 8-1)** and standard hydraulics calculations.

$$Q = 1.982LH^{1.5} \qquad \textbf{(Equation 8-1)}$$

Where:

Q	=	Flow over the weir (m³/s)
L	=	Width of the notch (m)
H	=	Flow depth over the weir (m)

In addition, a number of conditions are required to ensure acceptable measurement accuracy. These conditions are as follows, and are depicted in **Figure 8.2**:

- The allowable margin of error between the lengths of notch 1 (L1) and notch 2 (L2). L2 must be smaller than 4.5 times L1 to ensure a percentage error of less than 10% for a flow depth between these notches of 300 mm.
- The water depth upstream of the weir (Po) must be larger than half of the energy level above the crump, plus 50 mm.
- The downstream water level (hb) must be less than 90% of the upstream water level (ha). All heights are measured above the crest of the crump weir.
- The position of the weir must be such that the Froude number of the water upstream of the weir is less than 0.4.

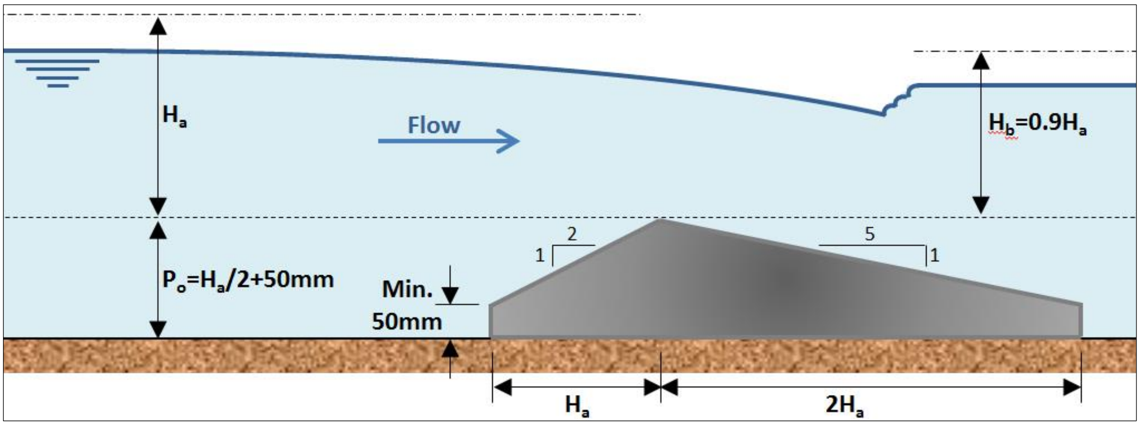

Figure 8.2: Recommended design dimensions and annotation of gauging weir

8.2.5 Weir layout

It is important for a division wall to be constructed between each notch level. This assists in preventing siltation and improves the accuracy of the gauging weir. These division walls must be extended to six times the energy level above the crest, upstream of the weir. Plan and upstream views of each gauging weir are included in **Annexure 8 C** as **Figure 8.C.1, Figure 8.C.2** and **Figure 8.C.3**.

8.3 WEIR 1: UPSTREAM OF SMITHFIELD DAM

8.3.1 Positioning of location 1 site

Three potential sites within the first 1.5 km immediately downstream of the proposed Impendle Dam site were investigated; however, one of these was discarded as there was a stream flowing directly onto the site. The other two were investigated regarding access, river characteristics and available materials and foundation and are discussed in **Table 8.1**.

Table 8.1: Weir 1: Upstream of Smithfield Dam selection of Site 1

Location of Weir 1: Upstream of Smithfield Dam	Site 1	Site 2
Coordinates	29°39'8.92"S; 29°46'29.65"E	29°39'23.79"S; 29°46'48.66"E
Access	Good quality dirt roads providing access on right bank of river. Roads are approximately 100 m from river.	Good quality dirt roads providing access on right bank of river. Roads are approximately 100 m from river.
Exposed Rock in River	Substantial amounts of unweathered rock daylight on both the banks and in the channel.	Substantial amounts of unweathered rocks daylight on both the banks and in the channel.
Length of Straight Pool Upstream of Weir	140 m	350 m
Downstream Inundation potential During Low Flows	Aerated water present at the site and an increase in bed slope immediately downstream of the site.	Aerated water present at the site and an increase in bed slope downstream of the site.
Influence by Upstream Stream/Rivers	There are no streams entering the river between the proposed Impendle Dam site and this weir site.	There is a substantial stream entering the river between site 1 and 2, which a catchment area of approximately 6.3 km². This would distort the Impendle Dam discharge readings.

For the reasons mentioned in the table above, site 1 was the selected site for the weir upstream of Smithfield Dam.

8.3.2 Backwater analysis

The data produced through the use of HEC-RAS has been included as graphs depicting the upstream area and top widths for varying water levels, as well as a downstream water level versus flow graph. These are shown in **Figure 8.3**, **Figure 8.4** and **Figure 8.5**.

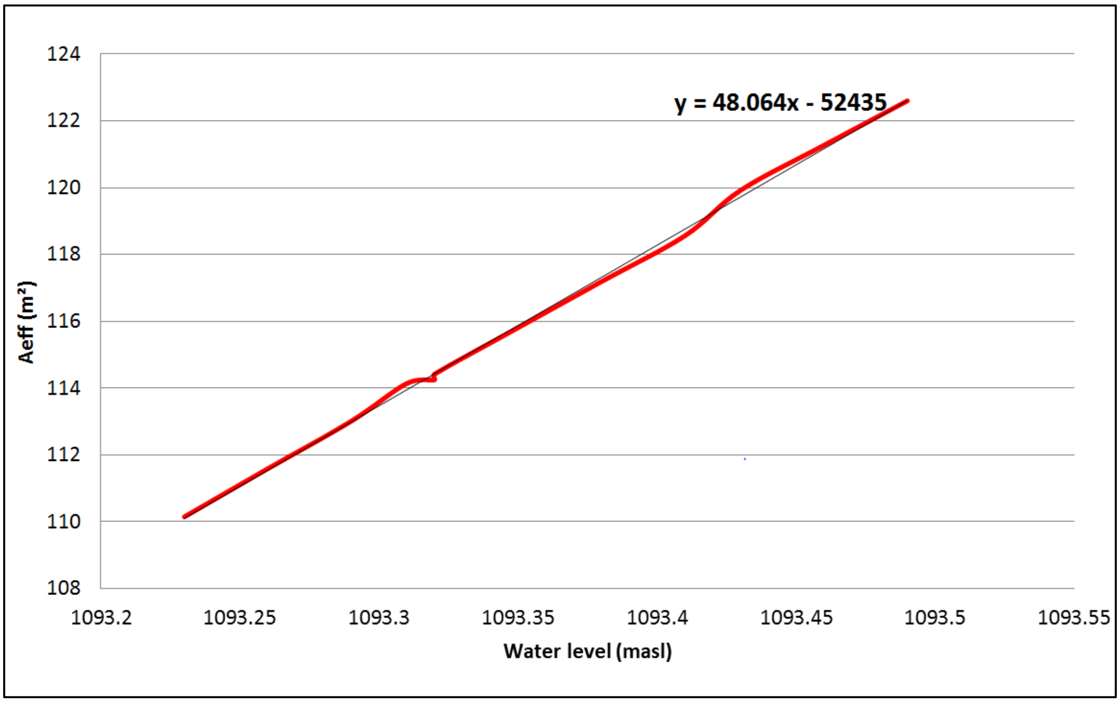

Figure 8.3: Section upstream of weir indicating the effective water area

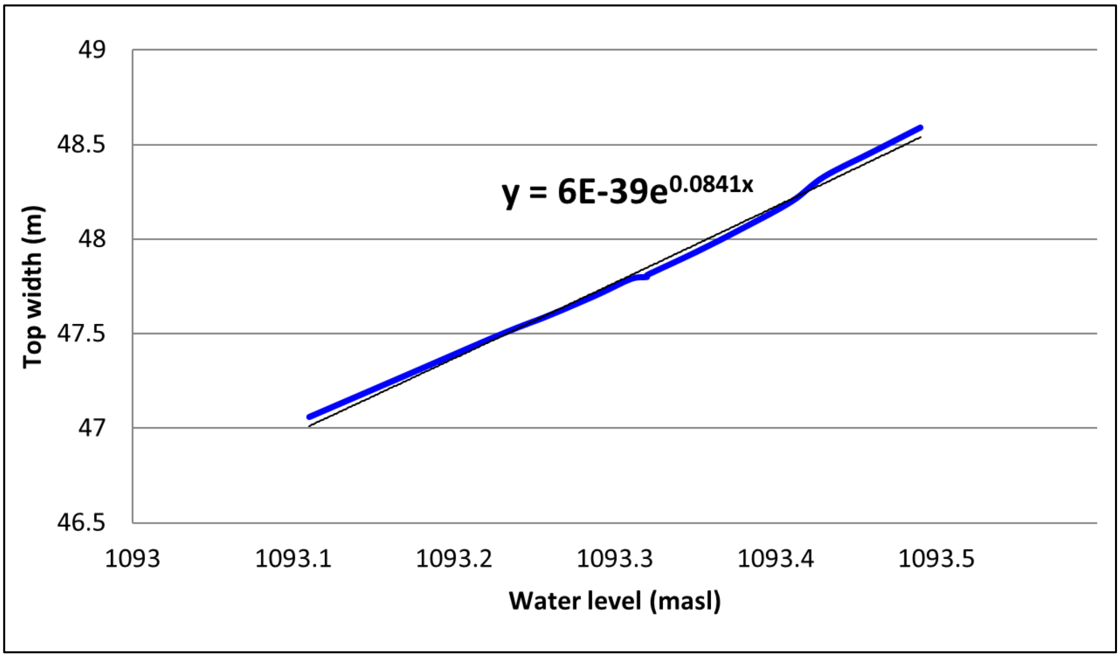

Figure 8.4: Section upstream of weir indicating water top width

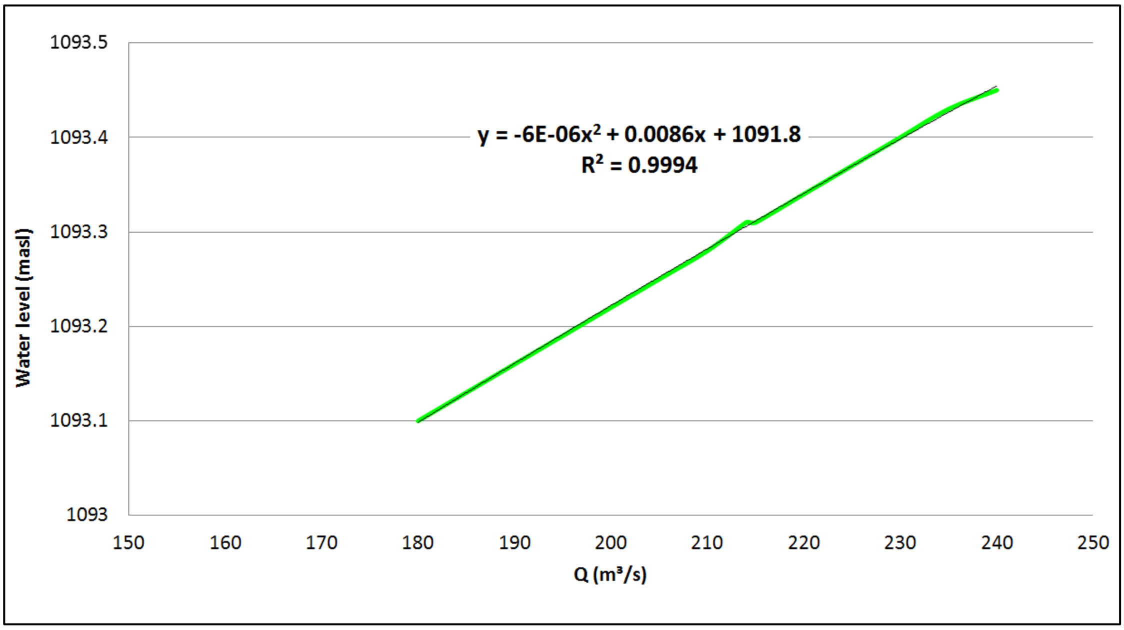

Figure 8.5: Section downstream of weir indicating water level versus flow

8.3.3 Flow measurement requirement

To determine the runoff volume for this site, the runoff volume used for the flow requirements of the third gauging weir site was transposed using the ratio of the MARs of the two sites. The naturalised MARs were determined during the hydrological analysis of the uMkhomazi system and form part of the uMWP.

The 95% and 75% runoff volumes were thus determined to be 468 m^3/s and 214 m^3/s respectively. The runoff volume of 214 m^3/s was used for design purposes.

8.4 WEIR 2: DOWNSTREAM OF SMITHFIELD DAM

8.4.1 Positioning of location 2 site

Two sites within the first 2 km downstream of Smithfield Dam were investigated regarding access, river characteristics and available materials and foundation and are discussed in **Table 8.2**.

Table 8.2: Weir 2: Downstream of Smithfield Dam selection of Site 1

Location of Weir 2: Downstream of Smithfield Dam	Site 1	Site 2
Coordinates	29°46'53.09"S; 29°55'52.70"E	29°47'11.91"S; 29°55'53.11"E
Access	There are roads close to the left hand side, but these would need to be extended by approximately 600 m and upgraded. Closest road on the right hand side is 900 m away and on top of a high bank.	Existing roads run near the site but would need to be extended by approximately 1.1 km. This extension would result in some destruction of virgin land.
Exposed Rock in River	Some unweathered surface rock boulders daylight on the banks; however, these are quite small (roughly 150 mm in diameter).	There is an exposed rock face on the right hand side banks upstream of the site. Aside from this there is limited rock exposure at the site.
Length of Straight Pool Upstream of Weir	250 m	150 m
Downstream Inundation potential During Low Flows	The flow velocity increases after the site, indicating an increase in the bed slope. This would assist in avoiding inundation at the site.	There is a section of aerated water downstream of the site which shows indications that there is also an increase in the bed slope.
Influence by Upstream Stream/Rivers	There is a small stream running into the river upstream of site 1. However, this stream is unavoidable as the site (at 1.3 km downstream of the proposed dam wall) could not be positioned above this stream.	There is an additional stream entering the river between site 1 and site 2. This stream has a slightly larger catchment than the one entering upstream of site 1. This may affect the accuracy of measuring discharge from Smithfield Dam.

Due to the stream that enters the river in between the two sites and the length of the straight pool leading up to site 2, site 1 was selected.

8.4.2 Backwater analysis

The information required for the design of the gauging weir and the graphs depicting the upstream area and top width as well as the downstream water level versus flow are shown in **Figure 8.6**, **Figure 8.7** and **Figure 8.8**.

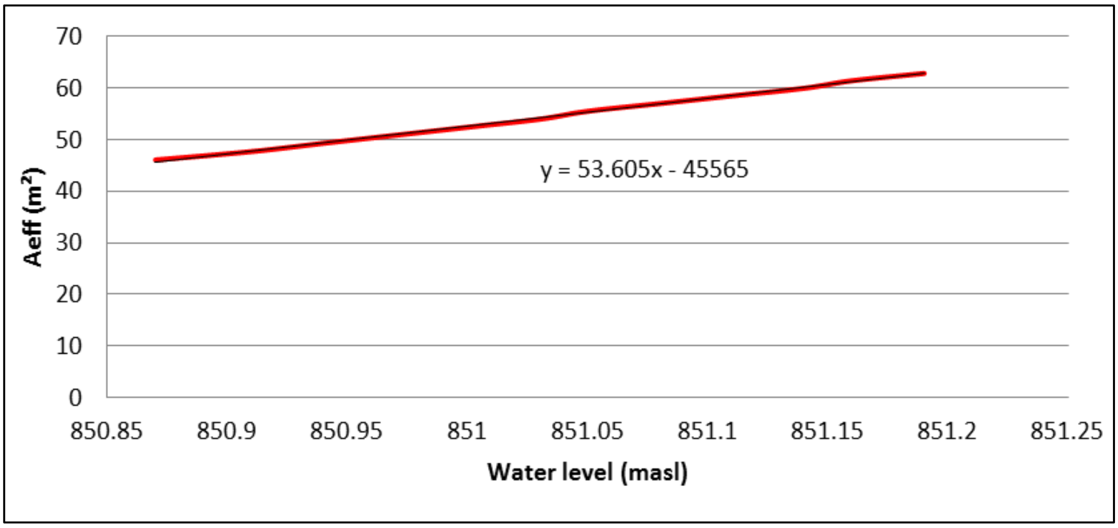

Figure 8.6: Section upstream of weir indicating the effective water area

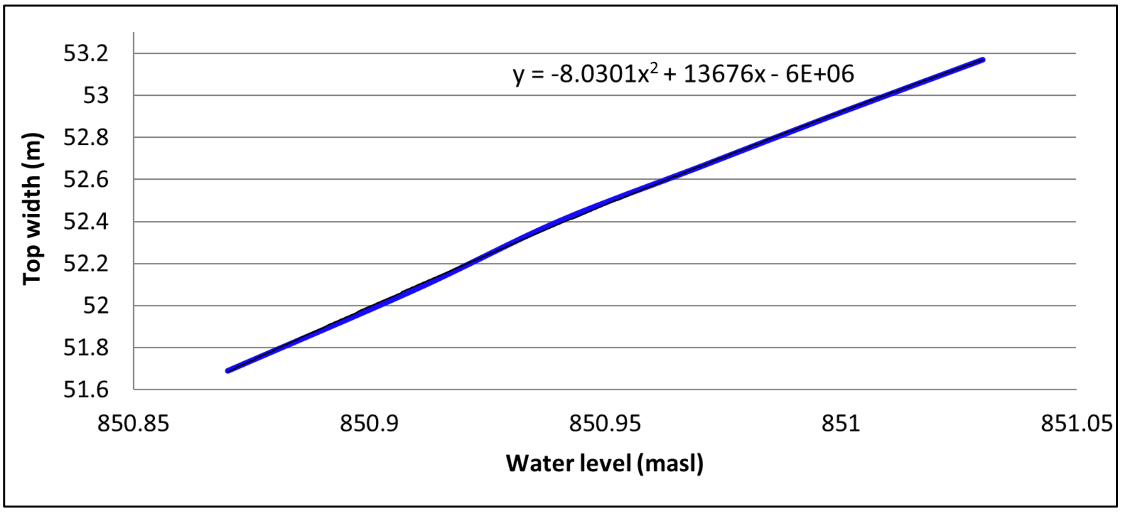

Figure 8.7: Section upstream of weir indicating water top width

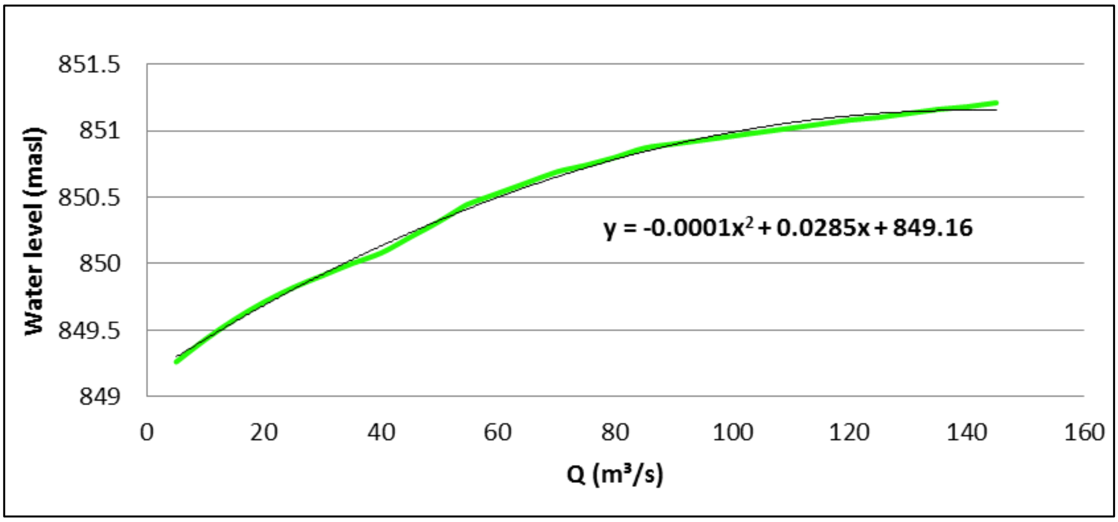

Figure 8.8: Section downstream of weir indicating water level versus flow

8.4.3 Flow measurment requirement

A spillway can measure flows accurately when the flow depth over the spillway is greater than 300 mm. Because the gauging weir is located immediately downstream of Smithfield Dam, the maximum flow to be measured by the gauging weir is equal to the flow with a flow depth of 300 mm over the spillway at Smithfield Dam. This is described by **(Equation 8-2)** and calculated below.

$$Qmax = CLH^{1.5}$$

(Equation 8-2)

Where:

Q_{max} = Flow over the spillway (m^3/s)

C = Discharge coefficient

L = Length of the spillway (m)

H = Flow depth over the spillway (m)

$$Qmax = CLH^{1.5}$$

$$= 1.7(150)(0.3)^{1.5}$$

$$= 41.9\ m^3/s$$

8.5 WEIR 3: NEAR EWR/IFR2

8.5.1 Positioning of location 3 site

Three sites were investigated along the river near EWR/IFR2 regarding access, river characteristics and available materials and foundation and are discussed in **Table 8.3**.

The uMkhomazi River's selected EWR/IFR site 2, as part of the *Mkomazi IFR Study* (IWR Environmental, Mkomazi IFR Study; Acc No: 502-2010; BRN: 503, Class: U1/U2, Box: 113, 1998), is downstream of Site 3.

Table 8.3: Weir 3: Near EWR/IFR2 selection of Site 3

Location of Weir 3: Near EWR/IFR2	Site 1	Site 2	Site 3
Coordinates	29°54'26.72"S; 30° 5'39.28"E	29°55'2.93"S; 30° 5'28.76"E	29°55'12.31"S; 30° 5'14.26"E
Access	Immediately upstream of the main road bridge crossing the river. Good access.	Access roads would need to be extended; this would involve the clearing of some natural bush (approximately 50 m).	Approximately 2.5 km downstream of the bridge. Access roads would need to be extended; this would involve the clearing of some natural bush (roughly 100 m). Close to EWR/IFR2 monitoring point.
Exposed Rock in River	None in the river stream, but some unweathered rock daylights on the banks.	Considerable amounts of unweathered rock daylight in both the river channel and on the banks.	Considerable amounts of unweathered rock daylight in both the river channel and on the banks.
Length of Straight Pool Upstream of Weir	230 m	+200 m	Approximately 550 m
Downstream Inundation potential During Low Flows	River has a low gradient at this point, but there is a control point with a gradient increase about 200 m downstream of the site.	There is a section of aerated water at the site and an increase in the bed slope immediately after the site.	Increase in river bed slope at the point of the site. Indicated by aerated water and increased velocity.
Influence by Upstream Stream/Rivers	There are no visible streams entering the river near the site.	Two streams join the river downstream of site 1. It is important in this situation to include the flow of these streams as Smithfield Dam will be required to release sufficient water to ensure sufficient flow at EWR/IFR2.	

8.5.2 Backwater analysis

The information required for the design of the gauging weir is shown in the figures below. Graphs depicting the upstream area, top width, and downstream water level versus flow are shown in **Figure 8.9**, **Figure 8.10** and **Figure 8.11**.

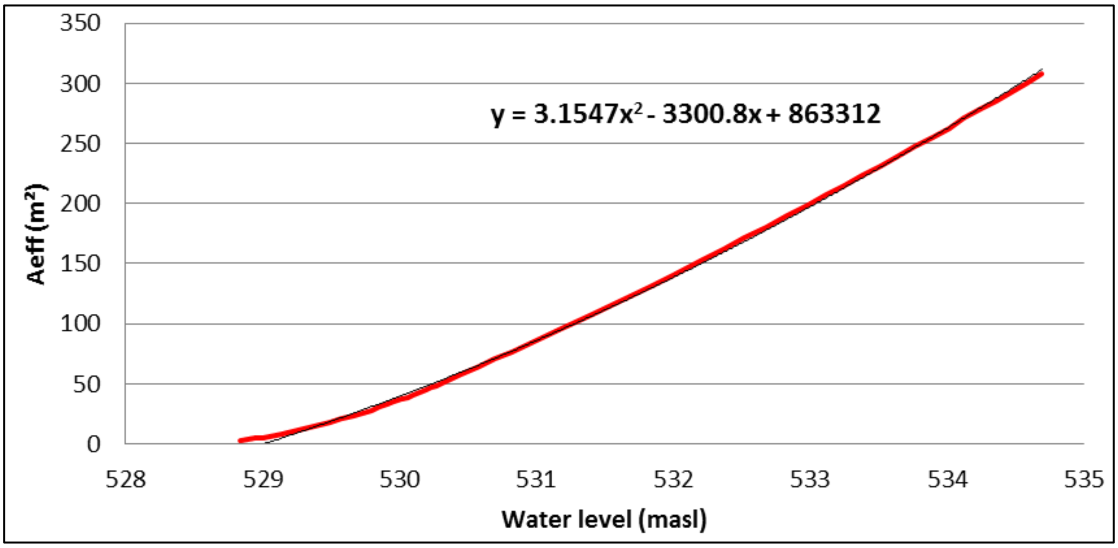

Figure 8.9: Section upstream of weir indicating the effective water area

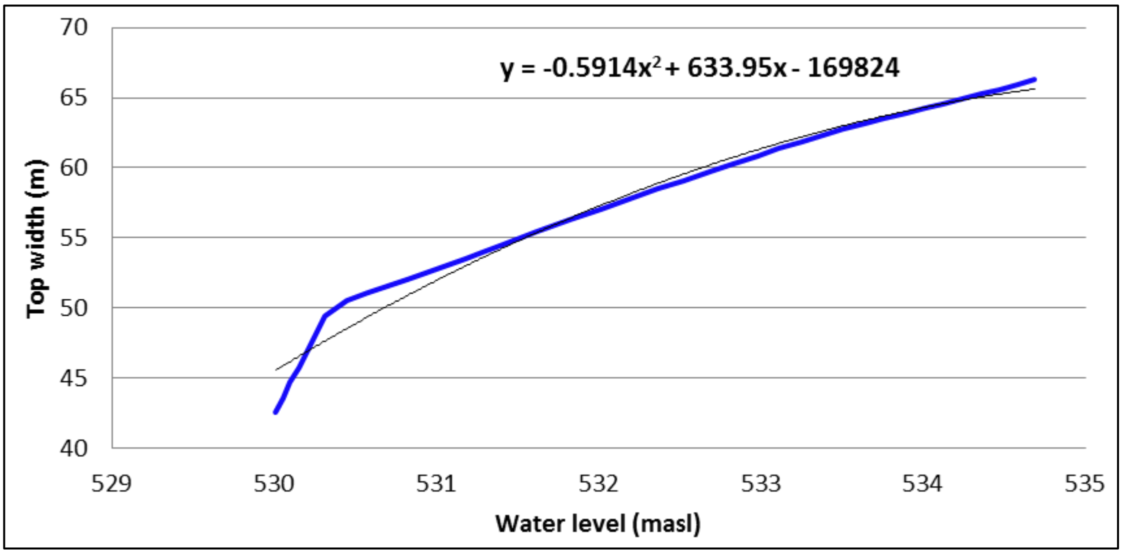

Figure 8.10: Section upstream of weir indicating water top width

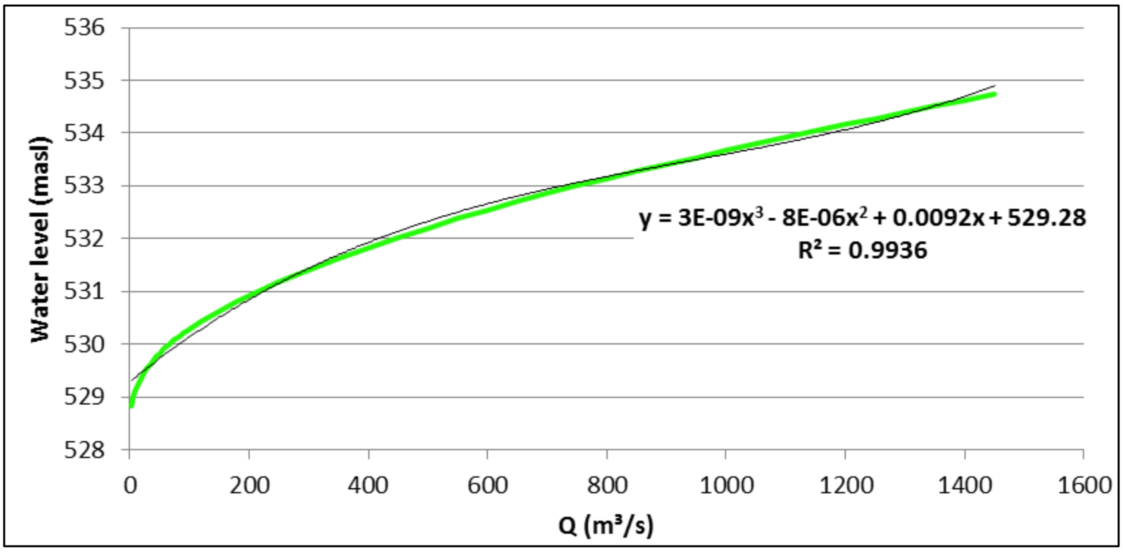

Figure 8.11: Section downstream of weir indicating water level versus flow

8.5.3 Flow measurment requirement

The runoff volume of the uMkhomazi River was determined at the gauging weir U1H005 and the volume transposed to the EWR/IFR2 site. The volume was determined by analysing the daily flows. The daily flows were patched and ranked form the highest to the lowest. The result of the runoff volume determination is presented in **Table 8.4** and **Figure 8.12**. With the fitting of a regression function (y = -90.92 ln (%) -1.2785) the discharge for 95% and 75% could be determined as 271 m^3/s and 124 m^3/s. These values were transposed to the larger catchment of the gauging weir with **(Equation 8-3)** and the required discharge at Location 3 determined as 596.2 m^3/s and 273 m^3/s.

The 95% runoff volume provided an impractical discharge for flow measurement and it was decided to use the 75% runoff volume of 273 m^3/s.

$$Qloc3 = Qgauge\sqrt{\frac{ALoc3}{AGauge}} \qquad \textbf{(Equation 8-3)}$$

Where:

Q_{Loc3} = Flow at Location 3 gauging weir (m^3/s)

Q_{gauge} = Flow as determined at gauge U1H005 (m^3/s)

A_{Loc3} = Catchment area of Location 3 gauging weir (1 744 km^2)

A_{Gauge} = Catchment area of gauge U1H005 (8 455 km^2)

Table 8.4: Stream flow volume determination

Ranked percentage (%)	Flows (m^3/s)	Cumulative flow per percentile (m^3)	Percentage stream volume (%)
1	189.8	4 195.691	13.2
5	81.9	11 588.870	36.5
10	50.7	16 514.100	52.1
20	27.2	22 179.380	69.9
30	16.9	25 519.090	80.5
40	10.8	27 622.040	87.1
50	7.3	29 005.510	91.5
60	5.2	29 956.280	94.5
30	3.8	30 651.810	96.7
80	2.8	31 154.010	98.2
90	1.9	31 517.180	99.4
95	1.3	31 642.820	99.8
99	0.6	31 705.810	100.0

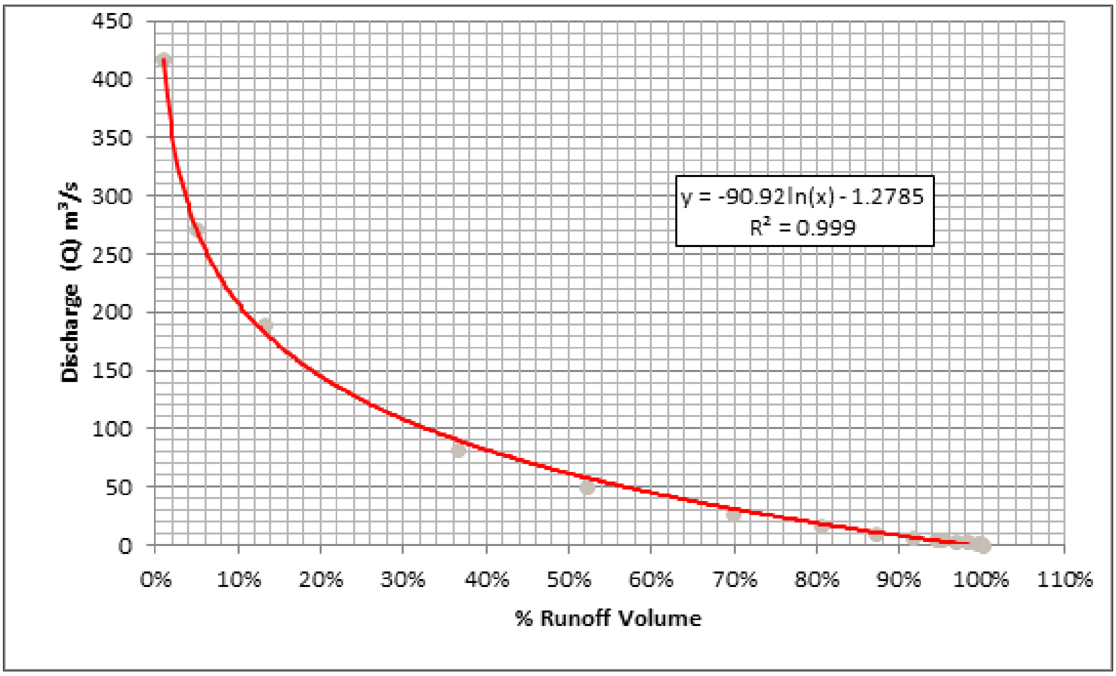

Figure 8.12: Percentage runoff volume versus discharge (m³/s) for gauge U1H005

8.6 Cost Estimate

A detailed cost estimate of all construction activities for each of the flow gauging weirs, comprising quantities and rates, has been completed, and is contained in **Annexure 8 B**. **Table 8.5** shows a summary of this cost estimate. Assumptions made in determining all cost estimates are described in **Section 14**. The total scheme cost estimate with all components added together is given in **Section 14.4**.

Table 8.5: Summary of cost estimate of activities for flow gauging weirs

Description	Cost (R million, excl. VAT)
Weir 1: Upstream of Smithfield Dam	9.2
Weir 2: Downstream of Smithfield Dam	8.3
Weir 3: Near EWR/IFR2	11.3
Miscellaneous	1.4
TOTAL	**30.2**

8.7 References

Van Heerden, Van Der Spuy and Le Roux, 1986. *Manual for the Planning, Design and Operation of River Gauging Stations TR 126*, Pretoria, South Africa: Department of Water Affairs (DWA).

9 ACCESS AND DEVIATION OF ROADS

9.1 GENERAL

The aim of this section is to investigate options for the realignment of various roads associated with the components of the uMWP, as well as route determination for new access roads to different elements of this project.

The following roads were identified, for which route determination is addressed in this report:

- Smithfield Dam
  - Deviation of the R617
  - Access road to Nonguqa
  - Access road to tunnel inlet portal
  - Access road to dam wall
  - Construction road
  - Main access road
- Tunnel
  - Access road to Ventilation Shaft 1
  - Access road to Ventilation Shaft 3
  - Access road to centre adit entry
- Langa Dam
  - Access road to tunnel outlet portal and Langa Dam (Option 1)
  - Access road to tunnel outlet portal and Langa Dam (Option 2)
- Gauging weirs
  - Access road to gauging weir 1
  - Access road to gauging weir 2
  - Access road to gauging weir 3

The layouts of the proposed options of the abovementioned roads for Smithfield Dam, Langa Dam, the tunnel and the gauging weirs are shown in **Annexure 9 A** as **Figure 9.A.1, Figure 9.A.2, Figure 9.A.3**, and **Figure 9.A.4** to **Figure 9.A.6**, respectively.

The objectives of the route investigation for the access roads were as follows:

- Establish a road centre line;
- Develop a preliminary longitudinal section;
- Quantify the scope of the construction works including pavement layers and fills;
- Investigate design criteria for stormwater drainage structures; and
- Estimate the design and construction cost.

A digital terrain model was created from a 5 m contour plan received from DWA. Topography information such as existing roads and water courses was provided in the form of images. This information was used for the conceptual design.

9.2 DESIGN STANDARDS

9.2.1 Geometric design standards

The approach was to apply horizontal, vertical and cross-sectional design standards to the permanent road which would accommodate a typical heavy haulage vehicle, rather than applying an arbitrary design speed. The geometric design standards used were therefore as follows:

- Minimum horizontal curve radius: 50 – 60 m
- Maximum grade: 14%
- Vertical curve sag K-Value: 8 (corresponds to 40 km/h)
- Vertical curve crest K-Value: 6 (corresponds to 40 km/h)

9.2.2 Design vehicle

The horizontal alignment and road width were checked by simulating an AASHTO WB-50 5-axle Semitrailer 16.76 m long.

9.2.3 Typical cross-section

The deviation of road R617 was designed as a paved road with a 3.5 m lane in each direction and a 1.0 m shoulder on either side, therefore a 9 m formation. For the gravel roads, a width of 8 m is proposed. Typical cross sections of the paved and unpaved roads are shown in **Annexure 9 B** as **Figure 9.B.2** and **Figure 9.B.3**.

9.3 FACTORS AFFECTING ROAD ALIGNMENT

The following factors were taken into consideration during the alignment of the road:

- Geometric design standards as mentioned above;
- The 1:100 year floodline;
- Areas of steep natural cross-fall; and
- The alignment of existing Provincial Road R617.

9.4 GEOTECHNICAL INVESTIGATION

9.4.1 Geology

The area of interest is underlain by rocks of the Volksrust Formation of the Ecca Group, comprising shales (mudrocks) with sub-ordinate sandstones. The sedimentary strata are essentially horizontal, and largely undisturbed. Regional dips of 3 – 7 degrees were recorded, while locally steeper dips are recognised and are ascribed to the intrusion of dolerites. Three near-horizontal dolerite sills have intruded mainly concordantly into the sedimentary strata and are responsible for the narrow river valley at the dam site and the presence of good quality rock for concrete aggregate and rockfill. A few faults with throws of up to 10 m have been mapped and one dolerite dyke traverses the left flank quarry area.

9.4.2 Sources of construction material

It is likely that fills and the selected layers would be able to be constructed from material obtained from cut during road construction.

Material for the sub-base layer and wearing course will have to be obtained from borrow areas. Material may have to be modified by means of stabilization if required.

9.4.3 Slope stability

Due to the steep natural cross-falls, high cut and fill slopes are inevitable.

a) Cut slopes

A cut slope of 1V:1H has been applied throughout for the purposes of determining earthwork volumes, as the locations of hard material, where steeper slopes may be used, are not accurately known at this stage. Cut slopes for detail design will have to be analysed for stability using soil properties determined during a detailed geotechnical investigation.

b) Fill slopes

If a fill slope of 1V:1.5H is applied throughout, the fill toe in sections with a steep cross-fall becomes located at excessive distances from the road edge. This will result in a large footprint for the road which may be found to be environmentally unacceptable. The fill volume will be further increased by the benching operation which will be required on steep slopes.

9.5 PRELIMINARY PAVEMENT DESIGN

The following preliminary pavement designs formed part of this investigation for the different roads:

- Gravel roads
  - 150 mm Gravel wearing course (G6) compacted to 95% of modified AASHTO compaction;
  - 150 mm Selected subgrade (G9) compacted to 93% of modified AASHTO compaction; and
  - Roadbed preparation/fill (G10) compacted to 90% of modified AASHTO compaction.
- Surfaced road
  - 19/9.5 mm double seal or 30 mm asphalt;
  - 150 mm Base (G4) compacted to 97% of modified AASHTO compaction;
  - 150 mm Sub-base compacted to 95% of modified AASHTO compaction;
  - 150 mm Upper selected subgrade (G7) compacted to 93% of modified AASHTO compaction;
  - 150 mm Lower selected subgrade (G9) compacted to 90% of modified AASHTO compaction; and
  - Roadbed preparation/fill (G10) compacted to 90% of modified AASHTO compaction.

9.6 STORMWATER DRAINAGE

All of the access roads under discussion are situated in the catchment of the uMkhomazi River, characterised by steep, rocky terrain and mountain grassland with scattered bush. The soils are easily eroded so great care needed to be taken in the design of drainage structures. The average annual precipitation for this area is 810 mm.

The following stormwater design standards were used:

- Minor catchments
  - Pipe and box culverts: 1:2 year flood return period
  - Side drains: 1:2 year flood return period
- Major catchments
  - Low water bridges: 1:5 year flood return period
  - Major bridges: 1:50 year flood return period

9.7 ROUTES INVESTIGATED

9.7.1 Smithfield Dam

a) Deviation of the R617

Three alternative route options were investigated to deviate road R617 around or over the expected full supply level (FSL) water line of the dam, to ensure continuity of the road. These options are shown in **Figure 9.1**.

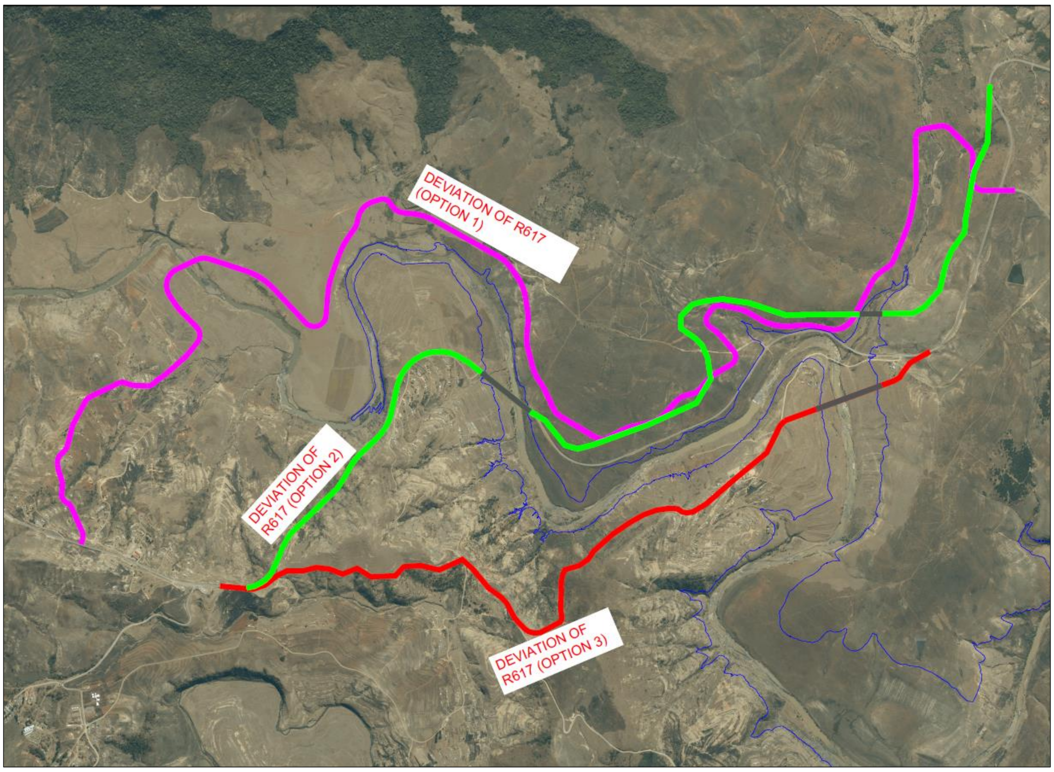

Figure 9.1: Alternative routes for deviation of the R617

Options 1 and 2 will deviate road R617, ensuring continuity of the surfaced route on the northern side of the dam, from a position at a village west of the dam wall to where it joins the surfaced section of this road on the northern side of the dam. Option 3 would also deviate on the western side of the dam, and will be south of the dam for the majority of the deviation, and end up on the north-eastern side of the dam.

As Options 2 and 3 will require long bridges over sections of the dam, these options were not further considered due to the high expense involved with bridges. Option 1 was therefore considered the most appropriate solution for the R617 deviation.

The preferred road for the deviation of road R617 will match the existing road which is a **7 m** wide surfaced road with a **3.5 m** lane in each direction including a 1 m shoulder on each side. The length of the deviation is **12.06 km** and has a maximum slope of **13.9%**.

b) Access road to Nonguqa

Three route options were considered to ensure access to Nonguqa, a village on the southern side of the dam, which is currently served by an unsurfaced road. Two of these options joined road R617 west of the dam. The third option considered is south of the dam. These are shown in **Figure 9.2**.

Option 1 was found to be the most suitable option, primarily due to the acceptable slopes of this road. This option runs along the southern end of the dam, first in an easterly direction when it leaves the R617, turning south after a distance of 2 km, and then east again until it reaches Nonguqa, which is situated in close proximity of the proposed dam wall.

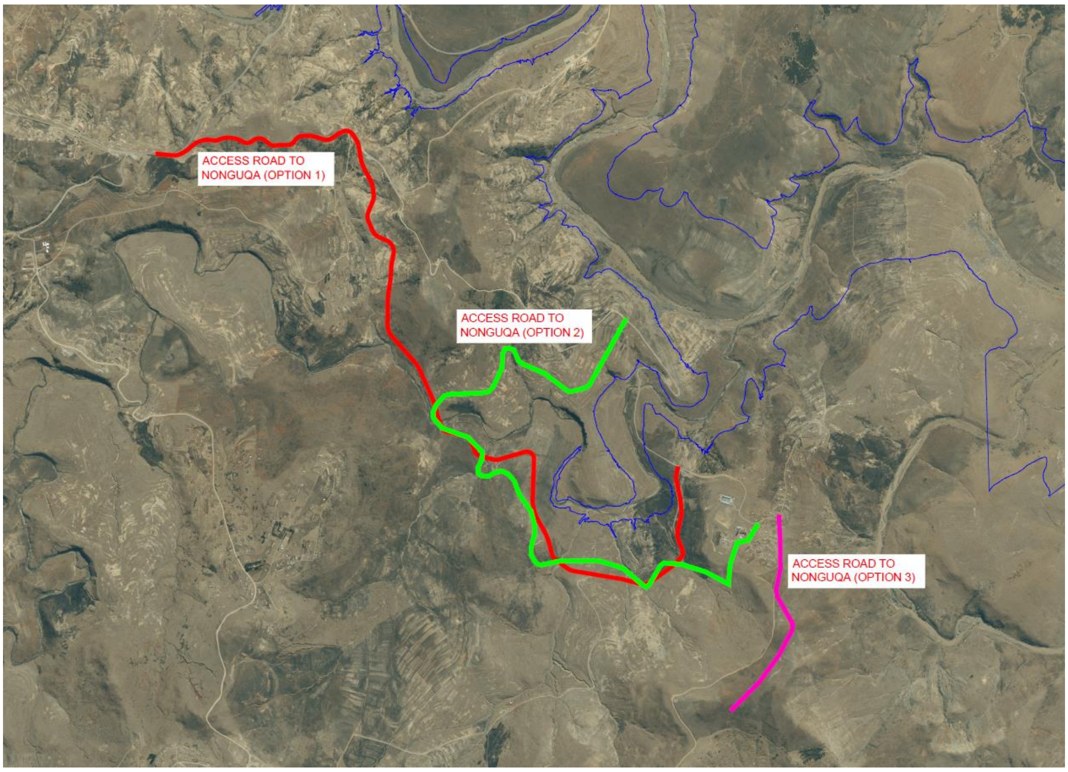

Figure 9.2: Alternative routes for deviation of the access road to Nonguqa

The length of the preferred deviation is **8.13 km** and it has a maximum slope of **12.8%**. The new route will have a gravel surface and a width of **8 m**, with a 2% camber from the centre line.

c) Access road to tunnel inlet portal

As with the access road to the intake tower to the dam, the same route applies to the tunnel inlet portal.

This road has a length of **0.23 km**, and turns off the main access road 5.21 km from the R617. The maximum slope on this proposed alignment is **13.9%.**

d) Access road to dam wall

Access to the dam wall can be taken off the route of the main access road, at a position 6.68 km from road R617.

This road has a length of **1.56 km**, and the maximum slope on this proposed alignment is **13.8%.**

e) Construction road

The main access road and the construction road are located on the eastern side of the dam basin, and are mostly on the alignment of an existing route, except for a section (main access road), which will be above the FSL of the dam. The construction road, which is the existing alignment of the road, is an alternative that can be used as an access road during the construction of the dam.

The two routes investigated have a common alignment up to 7.0 km. Route 1 then continues in a southerly direction to link up with Road D874 while Route 2 turns eastward to link up with Road D874 (refer to **Figure 9.A.1** in **Annexure 9 A**).

This road has a length of **3.39 km**. The maximum slope on this proposed alignment is **10.6%.**

f) Main access road

The main access road is on the eastern side of the dam basin, mostly on the alignment of an existing route, except for a section almost parallel to the construction road, which will be above the FSL of the dam. The access road will give permanent access to the dam wall, intake tower and tunnel inlet portal.

This road has a length of **7.50 km**. The maximum slope on this proposed alignment is **11.16%.**

The main access road will initially be paved for use during construction, where after the road will be re-sealed at the end of the construction period to serve as a permanent access road.

g) Layout of the roads

The layout of the access roads at Smithfield dam is shown on **Figure 9.A.1** included in **Annexure 9 A** and **Figure 9.C.1** to **Figure 9.C.9** in **Annexure 9 C**.

h) Scope of construction work

The approximate earthworks volumes for all the roads at Smithfield dam are shown in **Table 9.1**.

i) Cost estimate

The cost estimate of the detail design and construction of the access road is shown in **Table 9.2**. It is assumed that material for layer works will be sourced from commercial sources. Further testing of materials from the dam basins can be done which if successful could provide a saving.

Table 9.1: Estimated earthworks volumes for access roads at Smithfield Dam

Description	Access Roads Material Volume (m³)					
	Deviation of R617	Nonguqa	Tunnel inlet portal	Dam wall	Construction road	Main access road
Wearing course and base course	16 279	9 759	271	1 874	4072.8	8 995
Gravel subbase	16931	10 126	281	1 945	4225.53	9 332
Selected layer	34 729	10126	281	1945	4223.53	9332
Cut to fill	250 000	28 811	1 565	6 866	34 198	25 402
Cut to spoil	60 000	43 827	-	-	-	48 251
Import required	-	-	439	4 321	10 764	-
Asphalt/double seal	84 413*	-	-	-	-	90 000*

**This figure represents area (m²), not volume*

These quantities were used to estimate the cost for these roads.

Table 9.2: Cost estimate for access roads at Smithfield Dam

Item	Description	Amount (R)					
		Deviation of R617	Nonguqa	Tunnel inlet portal	Dam wall	Construction road	Main access road
1	Accommodation of traffic	482 360	162 660	9 040	62 480	135 760	299 840
2	Clearing and grubbing	1 205 900	1 219 950	6 780	46 860	101 820	224 880
3	Cut to fill	20 000 000	2 593 051	125 229	549 270	2 735 840	2 032 178
4	Cut to spoil	4 500 000	3 944 470	-	-	-	3 618 839
5	Import	-	-	35 129	345 682	861 130	-
6	Extra over for excavation in hard material	17 500 000	1 750 000	14 000	525 000	525 000	525 000
7	Formation preparation	602 950	406 650	11 300	78 100	169 700	374 800
8	Selected layer	6 251 386	1 518 838	50 647	350 044	760 595	1 679 854
9	Sub-base	3 724 784	1 518 838	61 901	427 832	929 617	2 053 154
10	Wearing course/base course	4 558 302	975 960	81 360	562 320	1 221 840	2 698 560
11	Surfacing	16 882 600	-	-	-	4 887 360	21600000
12	Stormwater drainage	2 411 800	1 626 600	45 200	312 400	678 800	1 499 200
13	Bridges (1:5yr flood)	20 880 000	-	-	-	-	-
14	Erosion protection, landscaping & finishing	602 950	406 650	11 300	78 100	169 700	374 800
15	Erosion protection to high fill slopes	602 950	162 660	11 300	78 100	169 700	374 800
16	Road markings and road furniture	241 180	16 266	4 520	31 240	67 880	149 920
17	**Total for roadworks(1)**	**100 447 162(1)**	**16 302 592(1)**	**467 706(1)**	**3 447 427(1)**	**13 414 742(1)**	**37 505 824(1)**
18	Total cost per km(2)	8 329 643(2)	2 004 499(2)	2 069 494(2)	2 207 060(2)	3 952 487(2)	5 003 445(2)

(1)This cost is the activity cost only, and excludes Ps&Gs, Contingencies, VAT and professional fees.

(2)This is derived from the "Total for Roadworks" and must not be included when the total cost is calculated.

9.7.2 Tunnel

a) Access road to Ventilation Shaft 1

This road has a length of **70 m**. The maximum slope on this proposed alignment is **6.79%.**

b) Access road to Ventilation Shaft 3

This road has a length of **68 m**. The maximum slope on this proposed alignment is **13.64%.**

c) Access road to centre adit entry

Two alternative route options were investigated to provide access to the central adit entry of the tunnel. Option 1 approaches the adit entry from the east, and Option 2 approaches from the north. These options are shown in **Figure 9.3** below. Option 2 was selected as the appropriate route because of its acceptable slopes.

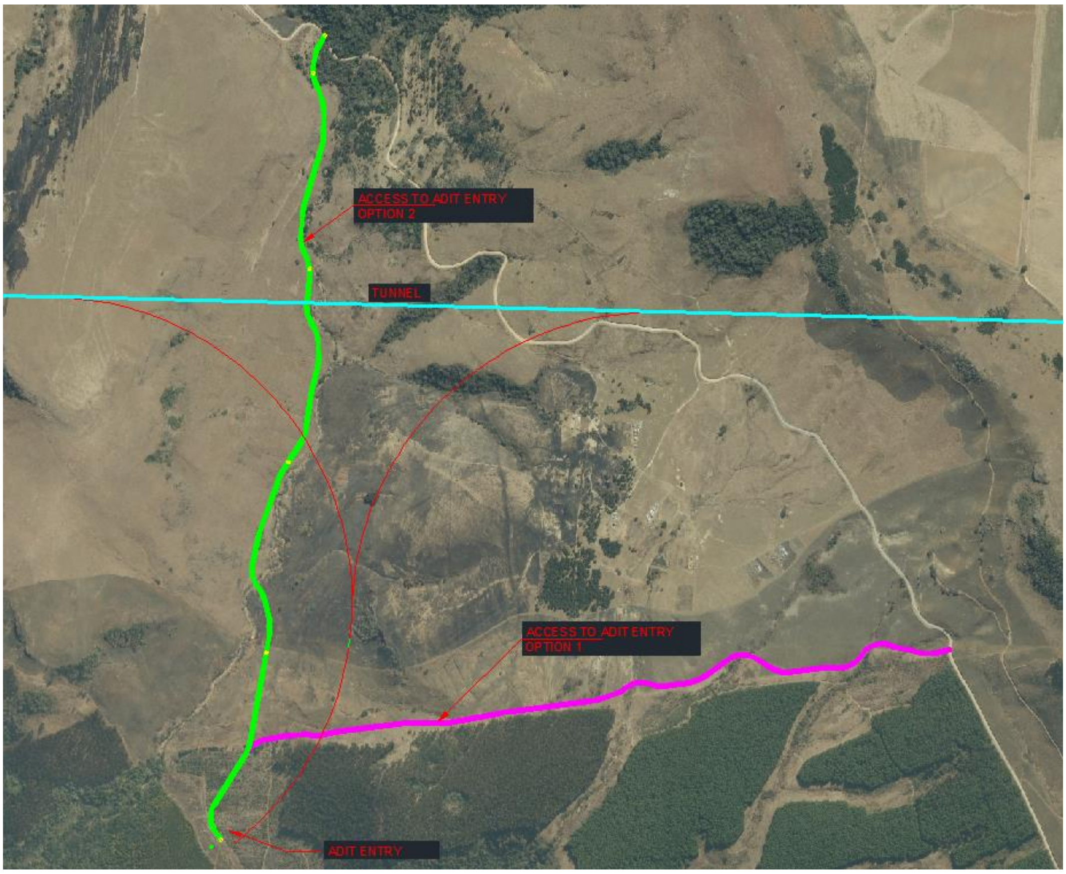

Figure 9.3: Alternative routes for access road to the tunnel centre adit entry

This road has a length of **2.10 km**. The maximum slope on this proposed alignment is **13.88%.**

d) *Layout of the roads*

The layout of the access roads at the Tunnel is shown on **Figure 9.A.3** in **Annexure 9 A** and in **Figure 9.C.10** and **Figure 9.C.11** in **Annexure 9 C**.

e) *Scope of construction work*

The approximate earthworks volumes for all the roads at the Tunnel are shown in **Table 9.3**.

Table 9.3: Estimated earthworks volumes for access roads at the Tunnel

Description	Access Roads Materials Volume (m³)		
	Ventilation Shaft 1	Ventilation Shaft 3	Centre Adit Entry
Wearing course and base course	2 522	82	2 522
Gravel sub-base	2617	85	2 617
Cut to fill	69.48	184	37 249
Cut to spoil	0	0	13 708
Import required	193.79	46	0

These quantities were used to estimate the cost for these roads.

f) *Cost estimate*

The cost estimate of the detail design and construction of the access road is shown in **Table 9.4**. It is assumed that sufficient amounts of material for layerworks will be obtainable by the roadworks contractor from nearby borrow pits.

Table 9.4: Cost estimate for access roads at the Tunnel

Item	Description	Amount (R)		
		Ventilation Shaft 1	Ventilation Shaft 3	Centre Adit Entry
1	Accommodation of traffic	84 080	2 720	84 080
2	Clearing and grubbing	63 060	2 040	63 060
3	Cut to fill	5 558	14 708	2 979 955
4	Cut to spoil	-	-	1 028 096
5	Import	15 503	3 686	-
6	Extra over for excavation in hard material	525 000	35 000	1 750 000
7	Formation preparation	105 100	3 400	105 100
8	Selected layer	471 058	15 239	471 058
9	Subbase	575 738	18 625	575 738
10	Wearing course/base course	756 720	24 480	756 720
11	Surfacing	-	-	-
12	Stormwater drainage	420 400	13 600	420 400
13	Bridges (1:5yr flood)	-	-	-
14	Erosion protection, landscaping & finishing	105 100	3 400	105 100
15	Erosion protection to high fill slopes	105 100	3 400	105 100
16	Road markings and road furniture	42 040	1 360	42 040
17	**Total for roadworks**	**3 274 458**	**141 658**	**8 486 447**
18	Total cost per km [(1)]	1 557 782[(1)]	2 106 400[(1)]	4 041 166[(1)]

(1)This is derived from the "Total for Roadworks" and must not be included when the total cost is calculated.

9.7.3 Langa Dam

a) *Access road to the tunnel outlet portal and Langa Dam*

Three options were investigated for this road, which are all shown in **Figure 9.4**. The latter section of Options 1 and 2 follows the same alignment for the last 2.3 km. From the earthworks volumes and the cost estimates, Option 2 is the preferred option between these 3 routes.

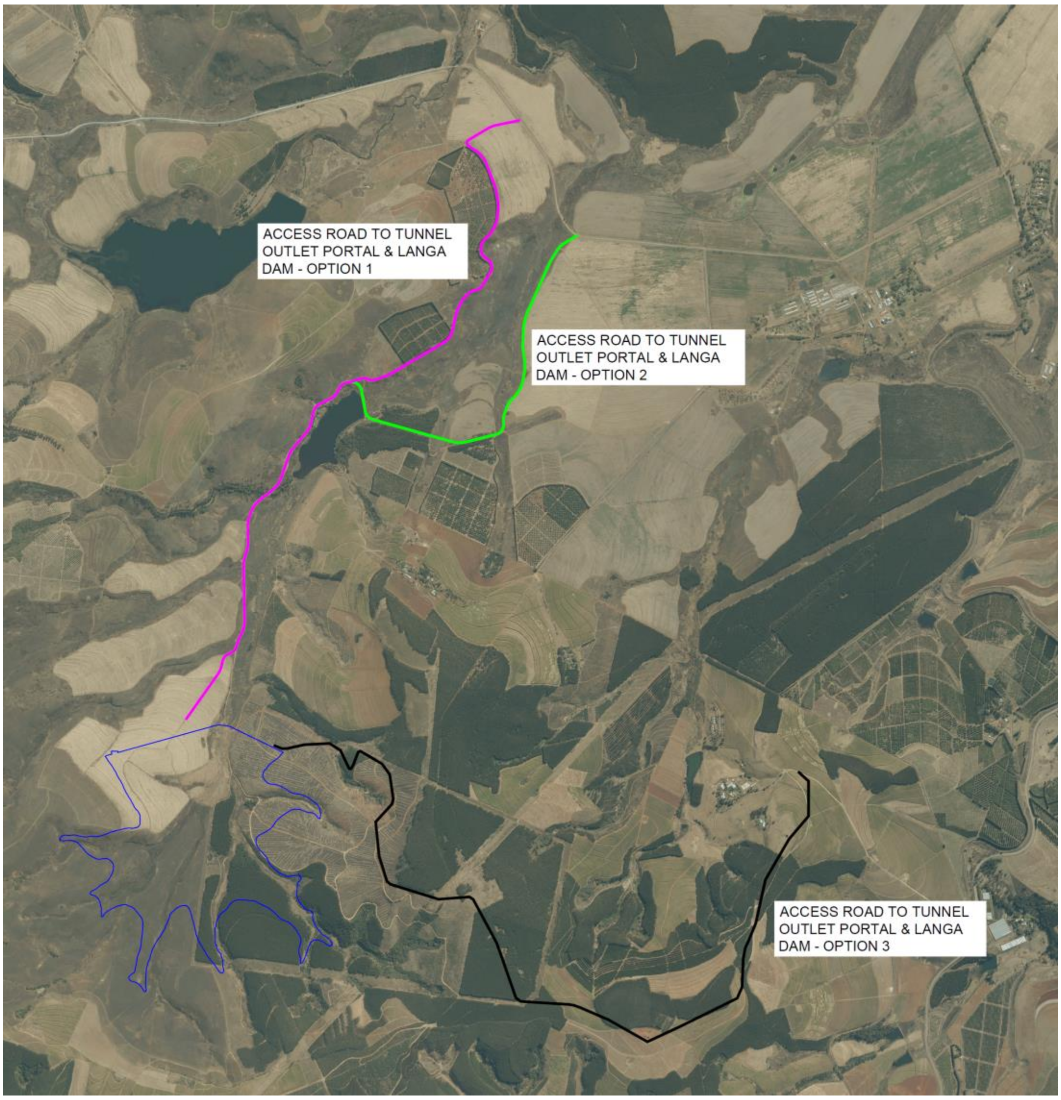

Figure 9.4: Alternative routes for deviation of the access road to the tunnel outlet portal and Langa Dam

The preferred road has a length of **4.69 km**. The maximum slope on this proposed alignment is **8.79%.**

b) *Access road to the WTW*

This road has a length of **2.37 km**. The maximum slope on this proposed alignment is **13.18%**.

c) *Layout of the roads*

The layout of the access road at Langa Dam is shown on **Figure 9.A.2** in **Annexure 9 A** and on **Figure 9.C.12** to **Figure 9.C.16** in **Annexure 9 C**.

d) *Scope of construction work*

The approximate earthworks volumes for the road at Langa Dam are shown in **Table 9.5**.

Table 9.5: Estimated earthworks volumes for access road at Langa Dam

Description	Access Roads Materials Volume (m^3)
	Tunnel outlet portal and Langa Dam
Wearing course and base course	5 628
Gravel sub-base	5 839
Cut to fill	21 943
Cut to spoil	5 847
Import required	-

These quantities were used to estimate the cost for these roads.

e) *Cost estimate*

The cost estimate of the detail design and construction of the access road is shown in **Table 9.6**. It is assumed that sufficient amounts of material for layerworks will be obtainable by the roadworks contractor from nearby borrow pits.

Table 9.6: Cost estimate for access road at Langa Dam

Item	Description	Amount (R)
		Tunnel outlet portal and Langa Dam
1	Accommodation of traffic	93 800
2	Clearing and grubbing	703 500
3	Cut to fill	1 974 870
4	Cut to spoil	526 230
5	Import	-

Item	Description	Amount (R)
6	Extra over for excavation in hard material	45 000
7	Formation preparation	7 035 000
8	Selected layer	875 858
9	Subbase	875 858
10	Wearing course/base course	562 800
11	Surfacing	15 758 400
12	Stormwater drainage	93 800
13	Bridges (1:5yr flood)	-
14	Erosion protection, landscaping & finishing	234 500
15	Erosion protection to high fill slopes	93 800
16	Road markings and road furniture	9 380
17	**Total for roadworks**	**28 882 795**
18	Total cost per km(1)	6 158 379(1)

(1)This is derived from the "Total for Roadworks" and must not be included when the total cost is calculated.

9.7.4 Gauging Weirs

a) *Access road to Gauging Weir upstream of Smithfield dam*

This road has a length of **0.170 km**. The maximum slope on this proposed alignment is **14%**.

b) *Access road to Gauging weir downstream of Smithfield dam*

This road has a length of **2.165 km**. The maximum slope on this proposed alignment is **8.3%**.

c) *Access road to Gauging Weir near the EWR/IFR2 site*

This road has a length of **2.516 km**. The maximum slope on this proposed alignment is **8.8%**.

d) *Layout of the roads*

The layout of the access roads to the Gauging weirs is shown on **Figure 9.C.4** to **Figure 9.C.6** included in **Annexure 9 C**.

e) *Scope of construction work*

The approximate earthworks volumes for all the roads at the gauging weirs are shown in **Table 9.7**.

Table 9.7: Estimated earthworks volumes for access roads to the gauging weirs

Description	Access Roads to Gauging Weirs Material Volume (m³)		
	Upstream of Smithfield dam	Downstream of Smithfield dam	Near IFR site
Wearing course and base course	204	2 598	3 019
Gravel sub-base	211	2 695	3 132
Cut to fill	307	8 513	26 656
Cut to spoil	0	0	1 184
Import required	97	247	0

These quantities were used to estimate the cost for these roads.

f) *Cost estimate*

The cost estimate of the detail design and construction of the access road is shown in **Table 9.8**. It is assumed that sufficient amounts of material for layerworks will be obtainable by the roadworks contractor from nearby borrow pits.

Table 9.8: Cost estimate for access roads to the gauging weirs

Item	Description	Amount (R)		
		Upstream of Smithfield dam	Downstream of Smithfield dam	Near EWR/IFR2 site
1	Accommodation of traffic	3 400	43 300	50 320
2	Clearing and grubbing	25 500	324 750	377 400
3	Cut to fill	27 630	766 170	2 399 040
4	Cut to spoil	0	0	106 560
5	Import	19 400	49 400	0
6	Extra over for excavation in hard material	70 000	350 000	525 000
7	Formation preparation	8 500	108 250	125 800
8	Selected layer	31 748	404 314	469 863
9	Subbase	31 748	404 314	469 863
10	Wearing course/base course	20 400	259 800	301 920
11	Surfacing	0	0	0
12	Stormwater drainage	3 400	43 300	50 320
13	Bridges (1:5yr flood)	0	0	0
14	Erosion protection, landscaping & finishing	8 500	108 250	125 800
15	Erosion protection to high fill slopes	3 400	43 300	50 320
16	Road markings and road furniture	340	4 330	5 032
17	**Total for roadworks**	**253 965**	**2 909 478**	**5 057 238**
18	Total cost per km (1)	1 493 912(1)	1 343 870(1)	2 010 032(1)

(1)This is derived from the “Total for Roadworks” and must not be included when the total cost is calculated.

9.8 Cost estimate of Access and deviation of roads

A detailed cost estimate of all construction activities for access and deviation of roads, comprising quantities and rates, has been completed, and is contained in **Annexure 9 D**. **Table 9.9** shows a summary of this cost estimate. Assumptions made in determining all cost estimates are described in **Section 14**. The total scheme cost estimate with all components added together is given in **Section 14.4**.

Table 9.9: Summary of cost estimate of activities for access and deviation of roads

Description	Cost (R million, excl. VAT)
Smithfield Dam	
Deviation of the R617	100.4
Access road to Nonguqa	16.3
Access road to tunnel inlet portal	0.5
Access road to dam wall	3.4
Construction road	13.4
Main access road	37.5
Tunnel	
Access road to Ventilation Shaft 1	3.3
Access road to Ventilation Shaft 3	0.1
Access road to centre adit entry	8.5
Langa Dam	
Access road to tunnel outlet and Langa Dam (Option 2)	28.9
Gauging Weirs	
Access road to gauging weir upstream of Smithfield Dam	0.3
Access road to gauging weir downstream of Smithfield Dam	2.9
Access road to gauging weir at EWR/IFR2 site	5.1
Miscellaneous	10.8
Total	**231.6**

9.9 FINDINGS AND RECOMMENDATIONS

- New access and construction roads as well as deviation of existing roads for the Smithfield Dam, tunnel, Langa Dam and the proposed gauging weirs were investigated for best alignment and allowable vertical slopes.
- No retaining structures are required on any of the routes, and an attempt was made to keep the fills to a minimum, also bearing in mind that at some sections, there will be huge volumes of spoil material which will be spoiled at the closest fill position and not at a spoiling site.
- It was assumed that road building material will be readily available from the quarries of the dams and therefore do not need to be imported from distant quarries or other commercial sources. Further investigations during tender and detail design phases would, however, be required to confirm the suitability of these materials. This study includes the importation of G5 materials from Midmar Quarry.
- The cost estimates for the roads include a 15% premium on actual construction cost for contractor's preliminary and general, as well as a 10% contingency allowance.
- The total estimated cost per km for gravel roads ranges between **R 2.89 million** and **R 5.14 million**, depending on the amount of earthworks required.
- The estimated cost per km for the surfaced R617 deviation is **R 8.3 million**, which includes 2 bridges and a **30 mm** asphalt wearing course.
- Detail topographical surveys must be done for all routes for tender and detail design purposes.

9.10 REFERENCES

AECOM, AGES, MMA & Urban-Econ, 2014. The uMkhomazi Water Project Phase 1: Module 1: Technical Feasibility Study: Raw Water; P WMA 11/U10/00/3312/3/1/10 – Write-up 4: Route investigation for various road alignments on the uMkhomazi-uMlaza transfer scheme, Pretoria, South Africa: Department of Water Affairs (DWA).

10 WASTE DISPOSAL SITES

Three waste disposal sites have been identified for disposal of construction materials during the construction of the uMWP1 and will form part of the EIA application. However, only two waste disposal sites – one near the tunnel inlet portal and one midway along the tunnel length near the tunnel access adits – will be used.

Mainly excavated material from the uMkhomazi – uMlaza Tunnel and the portals will be disposed at these sites. Tunnel muck and excavated material from the downstream outlet portal will be used for the construction of Langa Dam and thus the development of the third waste disposal site is not necessary.

The waste disposal sites will only be operational for the construction period of uMWP1 and will be rehabilitated afterwards.

10.1 VOLUME REQUIREMENT

The spoil volumes to be disposed of at the waste disposal sites and the sources thereof are summarised in **Table 10.1**. Large volumes of spoil from road construction will be spoiled at the closest fill position and not the designated waste disposal sites. Dolerite material excavated from the river diversion infrastructure such as from the inlet portal, outlet portal and tunnels, will be used as construction material for the Smithfield Dam main embankment.

A summary of the volume of waste to be disposed of at each of the waste disposal sites and the capacity thereof is included in **Table 10.2**.

Table 10.1: Spoil volumes, sources and disposal sites

Source	Excavated material, in-situ volume (BCM)	Excavated material (LCM[1])	Waste disposal site
Tunnel 1 inlet portal	365 000	584 000	Site 1
Tunnel 1 outlet portal	401 000	641 600	Langa Dam
Tunnel 1 (portion from central adit to inlet portal)	233 014	372 822	Site 2
Tunnel 1 (central tunnel section between adits)	32 558	52 093	Site 2
Tunnel 1 (portion from outlet portal to central adit)	285 117	456 187	Langa Dam
Tunnel 1 central access adit	79 334	126 934	Site 2
Tunnel 2 (first portion of tunnel)	1 590	2 544	Site 1
Tunnel 2 access adit	12 959	20 734	Site 1
Ventilation Shaft 1	216	346	Site 1
Ventilation Shaft 2	2 598	4 157	Site 2
Ventilation Shaft 3	3 593	5 749	Langa Dam
Roads	Approx. 137 791	-	Closest fill position

(1) Loose cubic metre based on a 1.6 swell factor

Table 10.2: Summary of waste disposal site volumes

Waste disposal site	Total excavated material to spoil (LCM)	Available volume at waste disposal site (LCM)
Site 1	607 624	615 000
Site 2	556 006	560 000
Langa Dam	1 103 536	See **Section 5**

10.2 LAYOUT OF THE SITES

The locations of the waste disposal sites are indicated in the general conveyance system layout (**Figure 4.1**).

Figure 10.1 and **Figure 10.2** show the layouts of waste disposal site 1 and waste disposal site 2 respectively.

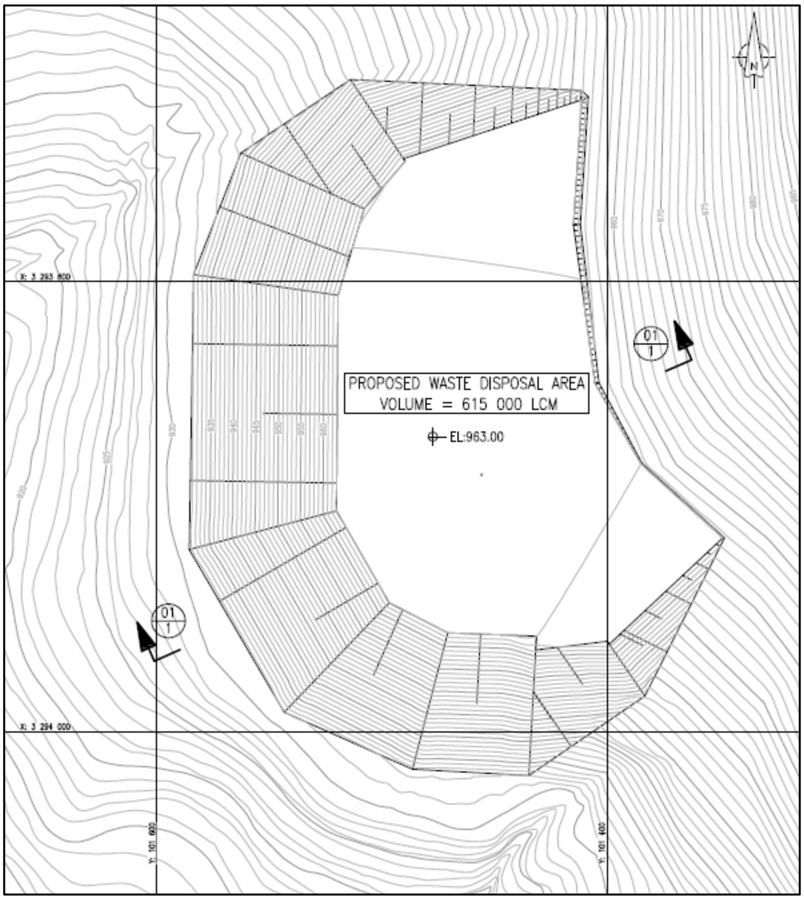

Figure 10.1: Waste disposal site 1

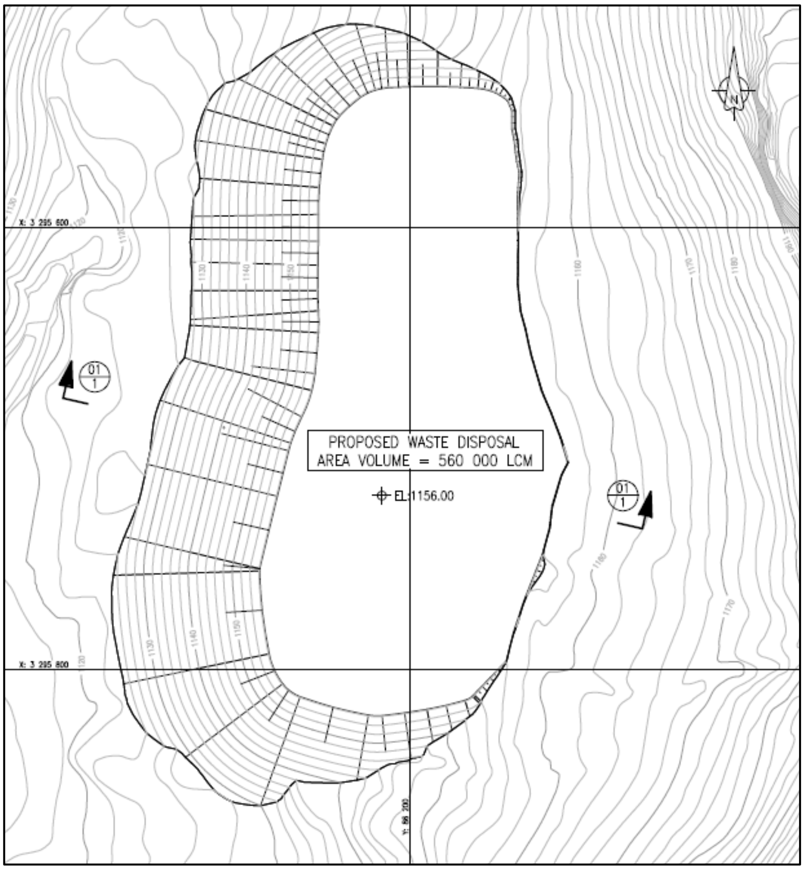

Figure 10.2: Waste disposal site 2

10.3 CLASSIFICATION

Disposed material will mainly be spoil from the uMkhomazi – uMlaza Tunnel and portal excavations which comprises of weathered and unweathered shale and dolerite. Other construction material such as concrete and earthfill will also be disposed of at the two waste disposal sites but to a lesser extent. The disposed material is considered to be categorised as (i) building and demolition waste not containing hazardous waste or hazardous chemicals and (ii) excavated earth material not containing hazardous waste or hazardous chemicals.

Waste will be disposed of at a maximum rate of approximately 600 ton per day, classifying the waste disposal site as a large landfill. The rate of disposal was calculated assuming the TBM is able to bore 0.492 km per month.

The potential for significant polluted leachate generation and the need for leachate management are considered negligible due to the nature of the disposed material.

With reference to the new *National Norms and Standards for the Disposal of Waste to Landfill* (Department of Environmental Affairs, 2013), the two waste disposal sites are classified as Class D landfills with Type 4 waste.

In order to adhere to the classification requirements, no unpermitted waste, such as domestic waste, may be disposed of at these sites. It is recommended that such waste be transported to and disposed of at commercial landfills in Pietermaritzburg or as arranged by the contractor.

10.4 LINING AND COVER

Class D landfills do not require a liner but a base preparation layer of reworked in-situ soil with a minimum thickness of 150 mm. The surface of the base preparation layer must be graded at a slope of 2% towards a central channel on the down gradient side of the waste disposal site from which sporadic leachate can be collected if it occurs. The central channel must contain a 150 mm layer of single-sized gravel or crushed stone to act as a finger drain.

The final cover of the waste disposal sites must be a 200 mm thick layer of topsoil lightly compacted after spreading and planted with local grasses and shrubs. Topsoil obtained from stockpiles of the material removed from the area of the waste landfill site and the reservoirs will be used.

Cross-sections of waste disposal site 1 and waste disposal site 2, as well as more detailed site layouts, are shown in **Figure 10.A.1** and **Figure 10.A.2** of **Annexure 10 A**.

10.5 Disposal of possible waste water

Waste water, generated by construction and equipment as well as groundwater expected to be encountered during the tunnelling process, require statutory permits to be discharged into surface water. All construction water will need to be treated before discharge into surface water.

Geotechnical investigation results indicated that water from one of the boreholes has high fluoride content. This necessitates that groundwater encountered during the construction of the uMkhomazi – uMlaza Tunnel must be evaluated for contaminates. Should the quality of the groundwater be inadequate for direct discharge into natural water courses, it must be treated prior to discharge.

The expected sources of groundwater and inflow rates must be identified prior to construction to provide adequate facilities for the removal of these waters from the construction area. If required, waste water lagoons or channels must be constructed to convey contaminated water to a treatment plant. The method and location of groundwater treatment, if necessary, will have to be confirmed during the detail design phase and are not included in this report; however, some costs have been allocated in this regard.

10.6 Cost estimate

A detailed cost estimate of all construction activities for the two waste disposal sites, comprising of quantities and rates, is included in **Annexure 10 B**. **Table 10.3** shows a short summary of the cost estimate pertaining to waste disposal site 1, waste disposal site 2 and miscellaneous items such as possible waste lagoons. Assumptions made in determining all cost estimates are described in **Section 14**. The total scheme cost estimate with all components added together is given in **Section 14.4**.

Table 10.3: Summary of cost estimate of activities for waste disposal

Description	Cost (R million, excl. VAT)
Waste Disposal Site 1	7.1
Waste Disposal Site 2	7.1
Miscellaneous	0.7
TOTAL	**14.9**

10.7 REFERENCES

RSA, 2013. *National Environmental Management: Waste Act, 2008: National Norms and Standards for Disposal of Waste to Landfill.* Pretoria, South Africa: Department of Environmental Affairs (DEA).

11 LAND ACQUISITION

Land is required for constructing the selected scheme. This section describes the required land and provides a cost estimate for acquiring the land. Both title deed and tribal land will need to be acquired for the project.

Section 64 of the *National Water Act (Act No. 36 of 1998)* enables the Minister of Water Affairs, or a Water Management Institution authorised by the Minister of Water Affairs in writing, to expropriate any property for any purposes contemplated by the National Water Act if the purchase is for public purposes or in public interest. Servitudes with specific purposes can also be registered.

The following approach is recommended for this project:

- Land inside purchase lines as well as areas at the dam walls must be expropriated for Smithfield and Langa Dams;
- Servitudes are required for protecting the tunnel from non-project related rights for the uMkhomazi – uMlaza Tunnel;
- Servitudes are required for maintenance and the right to provide water for the raw water pipeline, including the section to Langa Dam; and
- Land required for housing and other infrastructure required for the operation of the scheme is to be expropriated.

The areas and the methodology for determining the areas as well as cost estimates for the land to be acquired are described in this section and are shown on the drawings in **Annexure 11 A**.

11.1 SMITHFIELD AND LANGA DAMS

The purchase lines for dams in the Republic of South Africa are based on the 1:100 year recurrence interval backwater profile, up to the upstream point of no influence as per DWA policy. It is long standing DWA policy to add a buffer strip to the backwater profile for the 1:100 year recurrence interval. This buffer strip is the greater of the horizontal distance for a height of 1.5 m above the 1:100 year recurrence interval backwater level or 15 m horizontally from the 1:100 year recurrence interval backwater level.

The backwater levels for Smithfield and Langa Dams were calculated with the HEC-RAS model for their respective inflow hydrographs for the 1:100 year recurrence intervals. This was based on their respective FSLs of 930 masl and 923 masl, as well as spillway discharge rates at the 1:100 year flood.

The estimated areas of land that will be required for Smithfield and Langa Dams are 1 487.0 ha and 117.2 ha, respectively.

The purchase areas are shown in **Annexure 11 A** in **Figure 11.A.1** to **Figure 11.A.6**.

11.2 uMkhomazi – uMlaza Tunnel

The proposed servitude width for the tunnel for the purposes of this study is based on the following plan areas:

- The tunnel diameter plus 0.6 m on each side of the tunnel;
- Provision of a 5 m wide strip on one side of the tunnel for a service road; and
- Provision of a 2 m wide strip on the other side of the pipeline for additional working space.

A summary of the proposed servitude for the tunnel is given in **Table 11.1**, which is 24 m wide. The estimated area of the servitude that will be required for the tunnel, which is 32.5 km long, is 78 ha. An additional area of 10 ha was included for the inlet and outlet portal areas.

Table 11.1: Estimate of proposed servitude width for tunnel

Description	Servitude width (m)
Tunnel diameter	4.5
Provision for 0.6 m on sides of tunnel	1.2
Provision for service road	5.0
Provision for additional working space	1.5
Estimated servitude width	12.2
Rounded-off	12.0
Additional width	12.0
Final proposed servitude width*	**24.0**

* *Width assumed for feasibility planning, but final width to be determined when registering the servitudes*

11.3 BAYNESFIELD RAW WATER PIPELINE

The proposed servitude widths for the pipelines for the purposes of this study were based on the applicable guideline, and enhanced based on experience of construction practicality. The guideline used was the recommended excavation widths for pipelines by VAPS.

Two servitude widths were determined for each pipeline – temporary and permanent. The temporary servitude needed is wider than the permanent servitude, as sufficient room is needed for construction. This includes area for stockpiling topsoil and backfill material, additional width for the slopes of the excavated trench, and area for the movement of machinery that will lay the pipes. Provision was also made for a 5 m wide strip on one side of the pipeline for a service road, for both the temporary and permanent servitude.

Table 11.2 shows the proposed temporary and permanent servitude areas for both pipelines.

Table 11.2: Servitude widths for Tunnel – Langa Dam – Baynesfield Pipeline

Description	Pipeline 1 (2.6 m diameter)		Pipeline 2 (1.6 m diameter)	
	Permanent	Additional for temporary	Permanent	Additional for temporary
Length (m)	5 120		1 250	
Servitude width (m)	20	25	20	20
Servitude area (ha)	10.2	12.8	2.5	2.5

11.4 FLOW GAUGING WEIRS

The three proposed flow gauging weirs for the purposes of this project are:

- Weir 1: Upstream of Smithfield Dam (downstream of Impendle Dam);
- Weir 2: Downstream of Smithfield Dam; and
- Weir 3: Near EWR/IFR2, further downstream of Smithfield Dam.

The weir downstream of Impendle Dam is about 850 m downstream of the dam, and part of the land to be expropriated for the weir will be covered by the land to be expropriated for Impendle Dam. However, the land for the weir will need to be expropriated during Phase 1, and not only when Phase 2 begins. The gauging weir downstream of Smithfield Dam is about 1 km downstream of the dam, and can therefore not be included in the area to be expropriated for the construction of Smithfield Dam in future.

The proposed purchase lines for the gauging weirs are based on the backwater level for the design floods for each one of the weirs, plus a 15 m buffer zone.

The estimated areas of land (including servitudes) that will be required are 23.9 ha, 18.2 ha and 14.4 ha for the weir locations upstream of Smithfield Dam, downstream of Smithfield Dam, and at the EWR/IFR2 site, respectively.

11.5 ACCESS AND DEVIATION OF ROADS

The proposed width of the servitudes for the access roads is 12 m. A summary of the proposed 12 m wide servitudes for the roads is given in **Table 11.3**.

Table 11.4 summarises all the roads and their respective servitude areas. The total estimated area of the servitudes that will be required for the roads, with a total length of approximately 49.4 km, is 59.3 ha.

Table 11.3: Estimate of proposed servitude width for access roads

Description	Servitude width (m)
Road width	7.0
Provision for 2.5 m wide shoulders	5.0
Final proposed servitude width	**12.0**

Table 11.4: Estimate of proposed servitude areas for access roads

Road	Length (km)	Servitude area (ha)
Smithfield		
Deviation of R617	12.06	14.5
Nonguqa	8.13	9.8
Tunnel inlet portal	0.23	0.3
To dam wall	1.56	1.9
Construction road	3.39	4.1
Main access road	7.50	9.0
Tunnel		
Ventilation Shaft 1	0.07	0.1
Ventilation Shaft 3	0.07	0.1
Central adit entry	2.10	2.5

Road	Length (km)	Servitude area (ha)
Langa		
Tunnel outlet portal and Langa Dam	7.06	8.5
Flow gauging weirs		
Upstream of Smithfield Dam	0.17	0.2
Downstream of Smithfield Dam	2.17	2.6
Near EWR/IFR2 Site	2.52	3.0
Total	**47.03**	**56.6**

11.6 Relocation of Housing and Infrastructure

Certain housing and infrastructure will be inundated when Smithfield Dam is at its FSL, and so will need to be relocated to a position safely outside of the dam basin. The housing relocation comprises approximately 30 dwellings, and the infrastructure includes a package WTW. The cost of relocating this infrastructure is included in the environmental, landscaping and social costs item which is shown in **Table 14.4** in **Section 14.4**.

11.7 Cost Estimate

The cost estimates of land acquisition and expropriation, for the purposes of this study, are summarised in **Table 11.5** below. The detailed cost estimate, including quantities and rates for each land type, is given in **Annexure 11 B**. Assumptions made in determining all cost estimates are described in **Section 14**. The total scheme cost estimate with all components added together is given in **Section 14.4**.

Table 11.5: Cost estimate of land acquisition and expropriation

Road	Area (ha)	Cost (R million, excl. VAT)
Smithfield Dam	1 487.0	29.7
Langa Dam	117.2	3.8
uMkhomazi – uMlaza Tunnel	88.0	0.9
Tunnel – Langa Dam – Baynesfield Pipeline	28.0	0.2
Flow gauging weirs	56.8	1.1
Access and deviation of roads	59.3	1.2
Total	**1 836.3**	**37.0**

11.8 REFERENCES

RSA, 1998. *National Water Act (Act 36 of 1998)*. Pretoria, South Africa: Department of Water Affairs and Forestry (DWAF).

12 ACCOMMODATION AND RELATED STRUCTURES

Accommodation during construction of the project and during the operation of the scheme will be required. The cost, layout and number of accommodation units during the construction phase will be provided for through the preliminary and general component of the project tender and will be proposed by the contractor.

The accommodation and related structures during the operation of the project will be determined by the requirements of the operating entity. The detail of these structures will be determined during the tender and detail design stages of this project.

An estimation of the accommodation and related structures is made in this section in order to be able to determine a total cost estimate of the project.

12.1 LANGA DAM

The accommodation and related structures at the WTW will be used to service Langa Dam. The WTW forms part of the Module 2 investigation and the accommodation and related structures will therefore be included under that module. However, the construction related structures will be positioned in and near the Langa Dam reservoir as shown in **Annexure 11 A** as **Figure 11.A.4.**

12.2 SMITHFIELD DAM

The accommodation and related structures requirement at Smithfield Dam, for the operational phase, is estimated to be:

- An office complex with two offices and amenities with a size of 50 m^2;
- An operator's house consisting of a three bedroom unit with a floor space of 220 m^2;
- Three workers' houses consisting of two bedrooms each with a floor space of 92 m^2;
- A boat store of 40 m^2;
- A workshop of 100 m^2; and
- A covered parking area for 5 cars.

The operational phase structures will be constructed in a similar position as used for the construction phase. The location of these structures is shown in **Annexure 3 F** as **Figure 3.F.1.**

12.3 COST ESTIMATES

The cost of the accommodation and related structures were obtained from the AECOM published *Africa Property and Construction Handbook 2013*. Escalation of 9% was used to 2014 and the total cost for these structures was determined as R 3 128 000, excluding VAT. This cost forms part of "permanent infrastructure" in the miscellaneous costs in the detailed BoQ for Smithfield Dam. The summary BoQ for the accommodation and related structures only is attached in **Annexure 12 A.**

12.4 REFERENCES

AECOM South Africa (Pty) Limited, 2013. *AECOM Property and Construction Handbook 2013. 26th edition*. South Africa.

13 POWER REQUIREMENTS

Power supply to the scheme would be required during two phases:

- During the construction phase: power at the construction site; and
- After construction of the works is completed: permanent power supply to the site for operational purposes.

This section of the report describes the construction and permanent power requirements for the project. The basis followed to determine the power requirements was to abstract information from other contracts e.g., Spring Grove Dam (BKS (Pty) Ltd, 2008), Mohale Dam (Mohale Consultants Group, 1998), TBM tenders (Balci, et al., 2009) and manual calculations.

Figure 13.1 shows the general layout of the scheme and the location of the connection points for construction and permanent power supply.

13.1 CONSTRUCTION POWER REQUIREMENTS AND LOCATION

13.1.1 Description of power reqirements

To perform all various construction works, temporary power is required at the construction areas. The estimated power requirements for the various components of the project during construction are shown in **Table 13.1**. The table also indicates the latitude and longitude of each of the connection points for power supply.

Table 13.1: Power requirements and locations during construction

Component	Power requirement (kVA)	Location of connection points: Coordinates
Smithfield dam: Main and saddle embankments	2 000	29°46’47.82’’S; 29°56’09.46’’E
Adit at centre of tunnel	1 500	29°46’49.92’’S; 30°06’21.29’’E
Tunnel entrance at Langa Dam	1 500	29°46’16.42’’S; 30°18’15.97’’E
Langa Dam	2 000	29°47’14.49’’S; 30°18’26.40’’E
TOTAL	**7 000**	

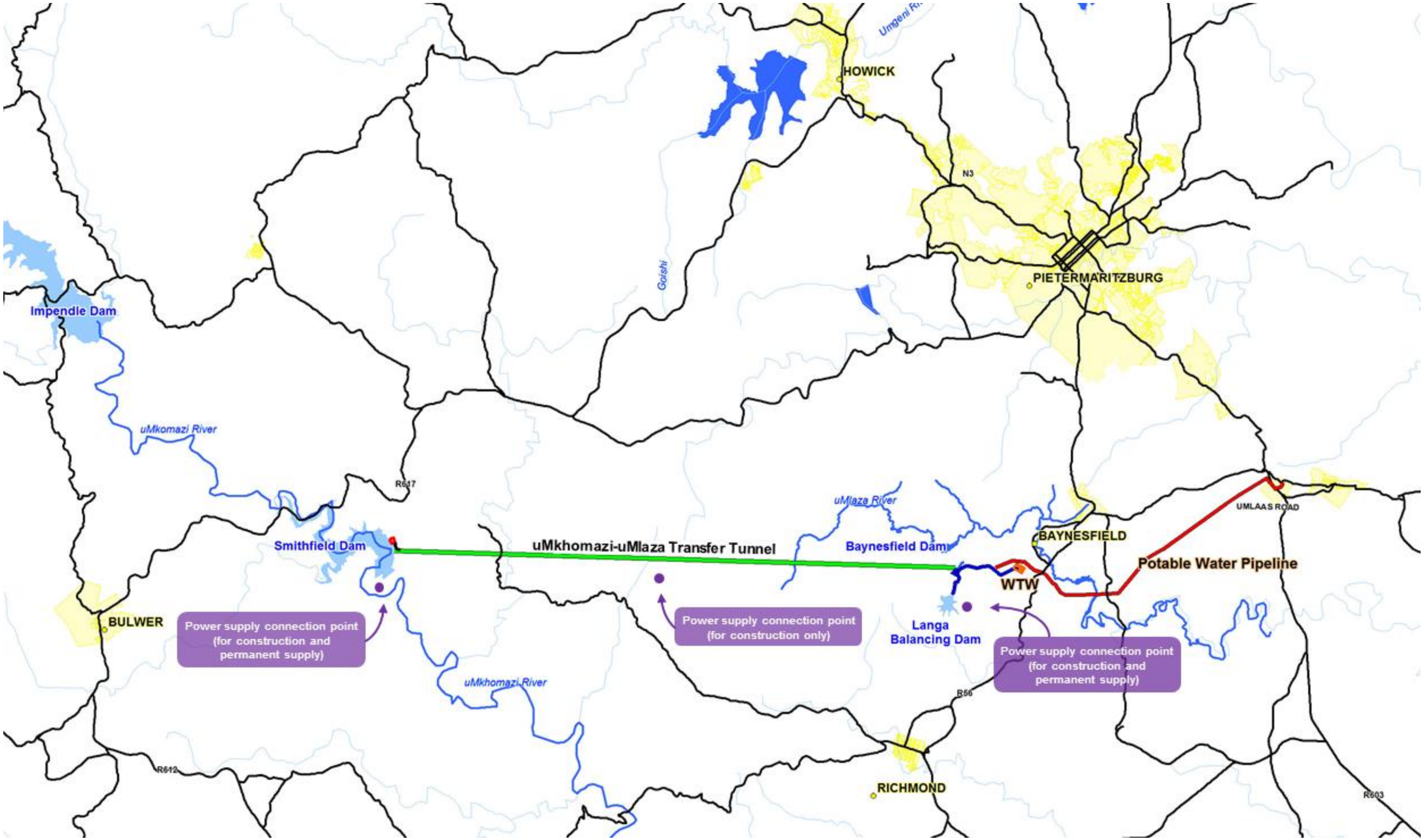

Figure 13.1: Scheme layout showing required locations for connection points for power

These requirements are needed for the operation of the various construction activities such as:

- Quarry operations;
- Main and saddle dam embankment;
- Spillway;
- Outlet works;
- Batch plants;
- Aggregate crushing plant;
- Concrete mixing plant;
- Laboratory;
- Contractor's and Engineer's offices;
- Operation village;
- Crane operations;
- Compressors;
- Conveyors;
- Lighting;
- Workshops;
- Precast yard for TBM segments; and
- Miscellaneous.

13.1.2 Summary of power reqirements

Table 13.2 below shows the recommended power supply connection points during the construction phase of the project. The difference between **Table 13.1** and **Table 13.2** is that there is no construction power supply connection point at the entrance to the tunnel at Langa Dam. Power will be supplied to the tunnel entrance from the power supply connection point at Langa Dam.

Table 13.2: Recommended locations for power supply connection points

Component	Power requirement (kVA)	Location of connection points: Coordinates
Smithfield Dam: Main and Saddle embankments	2 000	29°46'47.82''S; 29°56'9.46''E
Adit at centre of tunnel	1 500	29°46'49.92''S; 30°6'21.29''E
Langa dam	3 500	29°47'14.49''S; 30°18'26.4''E
TOTAL	**7 000**	

Once the construction of the dam is complete, the electricity supply to the site will be utilised for operation of the dam and the operator's housing requirements. This is described in detail further in this report.

13.2 PERMANENT POWER REQUIREMENTS

13.2.1 Description of power requirements

For operational purposes, power will be required permanently at both Smithfield and Langa Dam sites. Power at these sites is needed for sufficient operations and maintenance of the components of the project.

The operational power requirements at Smithfield and Langa Dam include the following components:

- Intake tower of the raw water tunnel (only at Smithfield Dam);
- Intake tower to the dam;
- Lights;
- Actuators for valves;
- Cranes for gates;
- A hydraulic power pack for operation of sleeve valves;
- Lifts inside intake towers;
- Permanent housing on site at Smithfield and Langa Dam;
- A site office and work shop; and
- Submersible pumps/de-watering pumps inside the dam.

All of the above components will receive electricity from the nearest connection point for permanent power supply, as can be seen in **Figure 13.1**.

The raw water tunnel intake tower requires approximately the same amount of power as needed for the operation of the intake tower of Smithfield Dam, except that there are more valves in the raw water tunnel intake tower and no hydraulic power pack. All lights are LED lights instead of traditional CFD lights. It is assumed that:

- Two floodlights of 50 W each are positioned above the crane on top of each intake tower; and
- Inside each intake tower, on every level, there are two 50 W lights to ensure enough light for safe movement for routine inspections.

The lift in each intake tower needs approximately 20 kW to operate. Emergency gates are to be used. To lift the 5 ton gates, a crane of a 20 ton rating is to be used. The total electrical demand is estimated to be approximately 20 kW. There are four valves in the intake tower at Smithfield Dam, 19 valves in the raw water tunnel intake tower and only one valve in the intake tower at Langa Dam. Each valve is controlled by an actuator which requires 1 kW to operate them.

Inside access tunnel 1, previously used as one of the river diversion tunnels, two 30 W lights are positioned every 10 m, one on each side of the tunnel. These are required for safe movement during routine checks and for general ease of movement inside the tunnels. A hydraulic power pack operates the two sleeve valves at the end of each of the pipes in the river diversion tunnel. One 1 kW power pack is required. Approximately 3 kW is needed for all de-watering pumps.

For lighting up the terrain of the Smithfield and Langa Dam sites, six 50 W lights will be strategically positioned. One site house will need approximately 8 kW of power and one office and work shop on site will need approximately 4 kW of power to satisfy the power requirements.

13.2.2 Summary of power requirements

Table 13.3 and **Table 13.4** show the permanent power requirements for Smithfield Dam and Langa Dam, respectively.

Table 13.3: Permanent power requirements at Smithfield Dam

Item no.	Component	Formula	Power requirement (kW)
1	Lights		
	Intake tower to outlet at dam	8 levels x 2 lights x 50 W	0.8
	Flood lights on crane	2 x 50 W lights	0.1
	Terrain	6 x 50 W lights	0.3
	Tunnel	2 lights x 400/10 m x 30 W	2.4
	Site houses		(Incl. in site houses)
	Site office and work shop		(Incl. in site office)
2	Crane	1 x 20 000 W crane	20.0
3	Actuators	4 x 1 000 W actuators	4.0
4	Hydraulic power pack	1 x 1 000 W power pack	1.0
5	Lift	1 x 20 000 W lift	20.0
6	Site houses	6 x 8 000 W	48.0
7	Site office	1 x 4 000 W	4.0

Item no.	Component	Formula	Power requirement (kW)
8	Intake tower at raw water tunnel inlet portal	0.8 + 0.1 + 20 + 19 + 20 + 3	62.9
9	Submersible pumps	3 000 W for pumps	3.0
Sub-total			**166.5**
Contingencies (20%)			**33.3**
Total			**199.8**
Recommendation			**250.0**

Table 13.4: Permanent power requirements at Langa Dam

Item no.	Component	Formula	Power requirement (kW)
1	Lights		
	Intake tower to outlet at dam	8 levels x 2 lights x 50 W	0.8
	Flood lights on crane	2 x 50 W lights	0.1
	Terrain	6 x 50 W lights	0.3
	Site houses		(Incl. in site houses)
	Site office and work shop		(Incl. in site office)
2	Crane	1 x 20 000 W crane	20.0
3	Actuators	1 x 1 000 W actuators	1.0
4	Hydraulic power pack	1 x 1 000 W power pack	1.0
5	Lift	1 x 20 000 W lift	20.0
6	Site houses	6 x 8 000 W	48.0
7	Site office	1 x 4 000 W	4.0
8	Submersible pumps	3 000 W for pumps	3.0
Sub-total			**98.2**
Contingencies (20%)			**19.6**
Total			**117.8**
Recommendation			**250.0**

13.3 CONCLUSION AND RECOMMENDATION

The power requirements during the construction phase of the project are higher than the requirements during the permanent phase. This is calculated using a power factor of 0.8. This means that a transformer with a capacity of 1 500 kVA (lowest transformer required for construction phase of project) is able to handle power requirements of up to 1 200 kW (1 500 kVA x 0.8). This is much higher than the required 250 kW recommended for permanent supply to each of the dam sites.

The same transformers, installed at Smithfield and Langa Dam, for the construction phase, will therefore be used to supply power permanently.

13.4 REFERENCES

Mohale Consultants Group, 1998. *Lesotho Highlands Water Project; Consulting Services for Mohale Dam; Stage 1 Services; Tender Design and Preparation of Tender Documents; Interim Design Report (Report no. 1017-1.7-02)*. Maseru, Lesotho: Kingdom of Lesotho Highlands Development Authority.

Balci, C., Tumac, D., Copur, H., Bilgin, N., Yazgan, S., Demir, E., et al., 2009. *Performance Prediction and Comparison with In-situ Values of a TBM: A Case Study of Otogar-Bagcilar Metro Tunnel in Istanbul*. Istanbul.

14 COST ESTIMATES

This section summarises the cost estimates of each of the raw water components of the technical feasibility study. Each of the components has been grouped as part of the following main infrastructure:

- Smithfield Dam;
- uMkhomazi – uMlaza Tunnel; and
- Langa Dam.

The reason for this grouping is so that an overall cost estimate would be determined for each likely tender contract, which includes the ancillary infrastructure. The ancillary infrastructure that is included is as follows:

- Flow gauging weirs;
- Roads;
- Hydropower plants;
- Transmission lines; and
- Waste disposal sites.

Assumptions which are applicable to all components are as follows:

- The base date that has been used for tariffs is ***March 2014***. Where costs have been escalated to this date, an escalation rate ranging between 6 and 9% has been used.
- All costs have been calculated excluding value added tax (VAT).
- Additional costs are as follows:
  - Miscellaneous: 5% of activity cost, unless specific items were listed;
  - Preliminary and general (Ps&Gs): 25% of activity cost;
  - Professional fees: 12% of activity cost;
  - Environmental, landscaping and social costs: 5% of activity cost, unless a more accurate estimate was known;
  - Contingencies: 25% of activity cost; miscellaneous; Ps&Gs; professional fees; environmental, landscaping and social costs; and land acquisition;
  - Implementing agent – TCTA: Lump sum based on estimate provided by TCTA; and

    - Implementing agent – Umgeni Water: 5% of activity cost; miscellaneous; Ps&Gs; professional fees; environmental, landscaping and social costs; and land acquisition.

- In determining activities required, various engineering specifications were used. In order to obtain a cost estimate for feasibility level purposes, these specifications were used broadly and in accordance with likely pay items, although specific pay items were not stated. The following specifications were used:

    - The Vaal Augmentation Planning Study (VAPS), which is a guideline for the preliminary sizing, costing and engineering economic evaluation of engineering options:

        - VAPS 6151 / Tunnel
        - VAPS 6149 / Rock

    - South African Bureau of Standards (SABS) – Standardized Specification for Civil Engineering Construction:

        - SABS 1200 C : Site clearance
        - SABS 1200 D : Earthworks
        - SABS 1200 DB : Earthworks (pipe trenches)
        - SABS 1200 DE : Small earth dams
        - SABS 1200 G : Concrete (structural)
        - SABS 1200 L : Medium pressure pipelines

    - South African National Standards (SANS):

        - SANS 10409 : Design, selection and installation of geomembranes

    - Department of Water Affairs – Specification DWS 0510 – Drilling and Grouting.

- Rates have been obtained from the following sources, where applicable:

    - Tender rates for the construction of Spring Grove Dam and Appurtenant Works, as part of the Mooi Mgeni Transfer Scheme Phase 2 (base rate from 2010); and
    - VAPS, as mentioned above (base rate from 1994).

14.1 Smithfield Dam

Table 14.1 provides a summary of the cost estimates for Smithfield Dam, including its associated infrastructure. The detailed cost estimates for each component are contained in several chapters of the annexure to this report.

Table 14.1: Summary of cost estimate for Smithfield Dam and associated infrastructure

Description	Cost (R million, excl. VAT)
River diversion works	178.5
Development of quarries and borrow areas	9.9
Smithfield Dam main embankment (zoned earth core rockfill dam)	813.5
Smithfield Dam saddle embankment (zoned earthfill dam)	252.1
Main embankment side channel spillway	189.7
Saddle embankment fuse plug spillway	66.0
Outlet works, intake structure	146.4
Tunnel intake structure	288.4
Access and deviation of roads	179.8
Flow gauging weirs	28.8
Waste disposal site 1	7.1
Transmission lines	5.0
Smithfield Dam HPP	36.6
Miscellaneous	86.05
TOTAL	**2 287.9**

14.2 uMkhomazi – uMlaza Tunnel

Table 14.2 provides a summary of the cost estimates for the uMkhomazi – uMlaza Tunnel and its associated infrastructure. The detailed cost estimates for each component are contained in several chapters of the annexure to this report.

Table 14.2: Summary of cost estimate for the uMkhomazi – uMlaza Tunnel and associated infrastructure

Description	Cost (R million, excl. VAT)
Transfer tunnel	3 362.2
Access and deviation of roads	11.9
Waste disposal site 2	7.1
Baynesfield HPP	42.8
Miscellaneous	542.2
TOTAL	**3 966.1**

14.3 LANGA DAM

Table 14.3 provides a summary of the cost estimates for Langa Dam and its associated infrastructure. The detailed cost estimates for each component are contained in several chapters of the annexure to this report.

Table 14.3: Summary of cost estimate for Langa Dam and associated infrastructure

Description	Cost (R million, excl. VAT)
River diversion works	1.4
Development of quarry	0.5
Langa Dam main embankment (concrete faced rockfill dam)	315.8
Spillway	3.6
Outlet pipes	12.8
Outlet works, intake structure	47.1
Baynesfield Raw Water Pipeline - 2.6 m diameter section	277.3
Baynesfield Raw Water Pipeline - 1.6 m diameter section	27.0
Access and deviation of roads	28.9
Miscellaneous	120.0
TOTAL	**834.3**

14.4 TOTAL COST FOR RAW WATER

Table 14.4 shows the summary of the total cost for the raw water system. Following that, **Table 14.5** shows a similar summary, but with all ancillary infrastructure costs assigned to their likely tender contracts as described in the introduction to this chapter.

Table 14.4: Summary of total cost estimate for the raw water system

Component	Cost (R million, excl. VAT)
Smithfield Dam	2 018
uMkhomazi – uMlaza tunnel	3 901
Langa Dam	439
Baynesfield Raw Water Pipeline	365
Transmission lines	5
Smithfield Dam and Baynesfield hydropower plants*	83
Waste disposal sites	15
Flow gauging weirs	30
Roads and bridges	232
Sub-total of activities	**7 088**
P&G costs (25% of activity cost)	1 772
Professional fees (12% of activity cost)	851
Environmental, landscaping and social costs (lump sum)	450
Land acquisition (lump sum)	37
Sub-total of activities and value-related costs	**10 198**
Contingencies (25% of above sub-total)	2 550
Implementing agent - TCTA (lump sum)	200
Total: Raw water system	**12 948**

**Included in total cost for raw water system for costing purposes*

Table 14.5: Summary of total cost estimate for the raw water system, grouped into likely tender contracts

Component	Cost (R million, excl. VAT)
Smithfield Dam	
Smithfield Dam	2 018
Access and deviation of roads	189
Flow gauging weirs	30
Waste disposal site 1	7
Transmission lines	5
Smithfield Dam HPP	38
Sub-total: Smithfield Dam	**2 288**

Component	Cost (R million, excl. VAT)
uMkhomazi – uMlaza tunnel	
uMkhomazi – uMlaza tunnel	3 901
Access and deviation of roads	12
Waste disposal site 2	7
Baynesfield HPP	45
Sub-total: uMkhomazi – uMlaza tunnel	**3 966**
Langa Dam	
Langa Dam	439
Baynesfield Raw Water Pipeline	365
Access and deviation of roads	30
Sub-total: Langa Dam	**834**
Sub-total of activities	**7 088**
P&G costs (25% of activity cost)	1 772
Professional fees (12% of activity cost)	851
Environmental, landscaping and social costs (lump sum)	450
Land acquisition (lump sum)	37
Sub-total of activities and value-related costs	**10 198**
Contingencies (25% of above sub-total)	2 550
Implementing agent - TCTA	200
Total: Raw water system	**12 948**

The above two tables include totals for all costs related to the construction of the infrastructure. However, part of the infrastructure will not be constructed by the DWS, and will therefore not be included in the institutional economic calculations relating to funding of the scheme; namely the hydropower plants. **Table 14.6** therefore shows the total cost estimate for the raw water system that will be used for institutional economic calculations.

Table 14.6: Summary of total cost estimate for the raw water system to be used in institutional economic calculations

Component	Cost (R million, excl. VAT)
Smithfield Dam	2 018
uMkhomazi – uMlaza tunnel	3 901
Langa Dam	439
Tunnel – Langa Dam – Baynesfield Pipeline	365
Transmission lines	5
Smithfield Dam and Baynesfield hydropower plants	Nil*
Waste disposal sites	15
Flow gauging weirs	30
Roads and bridges	232
Sub-total of activities	**7 005**
P&G costs (25% of activity cost)	1 751
Professional fees (12% of activity cost)	841
Environmental, landscaping and social costs (lump sum)	450
Land acquisition (lump sum)	37
Sub-total of activities and value-related costs	**10 084**
Contingencies (25% of above sub-total)	2 521
Implementing agent - TCTA (lump sum)	200
Total: Raw water system	**12 805**

**Not included as does not form part of raw water system. However, activity cost is R 83 million.*

14.5 CONSTRUCTION CASH FLOW FORECAST

This section describes the estimated cash flow forecast during the construction of uMWP-1. Three costs were considered, namely:

- The construction activity cost only.
- The construction activity cost, including costs for Ps&Gs and contingencies.
- The construction activity cost, including all additional costs, namely: Ps&Gs; professional fees; environmental, landscaping and social costs; land acquisition costs, contingencies and costs for an implementing agent.

Each sub-section to follow shows the total estimated capital cost for construction, with the estimated annual capital expenditure over the five years of the construction period. Each section also contains graphs showing the total annual and cumulative costs.

14.5.1 Cash flow: Activity cost only

Table 14.7 shows the cash flow forecast for construction, for the activity costs only.

Table 14.7: Construction cash flow forecast: Activity cost only

Item	Total capital cost (R million)	Capital cost per year (R million)				
		2018	2019	2020	2021	2022
		1	2	3	4	5
Raw water system						
Smithfield Dam	2 018	25	846	846	300	
Saddle embankment	318	25	147	147		
Main embankment	1 003		352	352	300	
All other components	696		348	348		
Langa Dam	439		30	204	204	
Main embankment	319		30	145	145	
All other components	119			60	60	
Tunnel	3 901	100	950	950	950	950
Raw water pipeline	365		50	158	158	
Access roads	232	116	116			
Gauging weirs	30		10	10	10	
Waste disposal sites	15	15				
Transmission lines	5	2.5	2.5			
Sub-total: Raw water system	**7 005**	**258**	**2 005**	**2 169**	**1 622**	**950**
Cumulative sub-total: Raw water system		**258**	**2 264**	**4 432**	**6 055**	**7 005**
Potable water system						
WTW	795			238	318	238
Potable water pipeline	1 143			343	457	343
Sub-total: Potable water system	**1 938**			**581**	**775**	**581**
Cumulative sub-total: Potable water system				**581**	**1 357**	**1 938**
Other components						
Baynesfield HPP	45				22	22
Smithfield HPP	38				19	19
Sub-total: Other components	**83**				**42**	**42**
Cumulative sub-total: Potable water system					**42**	**83**
Grand total	**9 025**	**258**	**2 005**	**2 750**	**2 439**	**1 573**
Cumulative grand total		**258**	**2 264**	**5 014**	**7 453**	**9 027**

Figure 14.1 depicts the above cash flow forecast for the total system (raw water, potable water and other components).

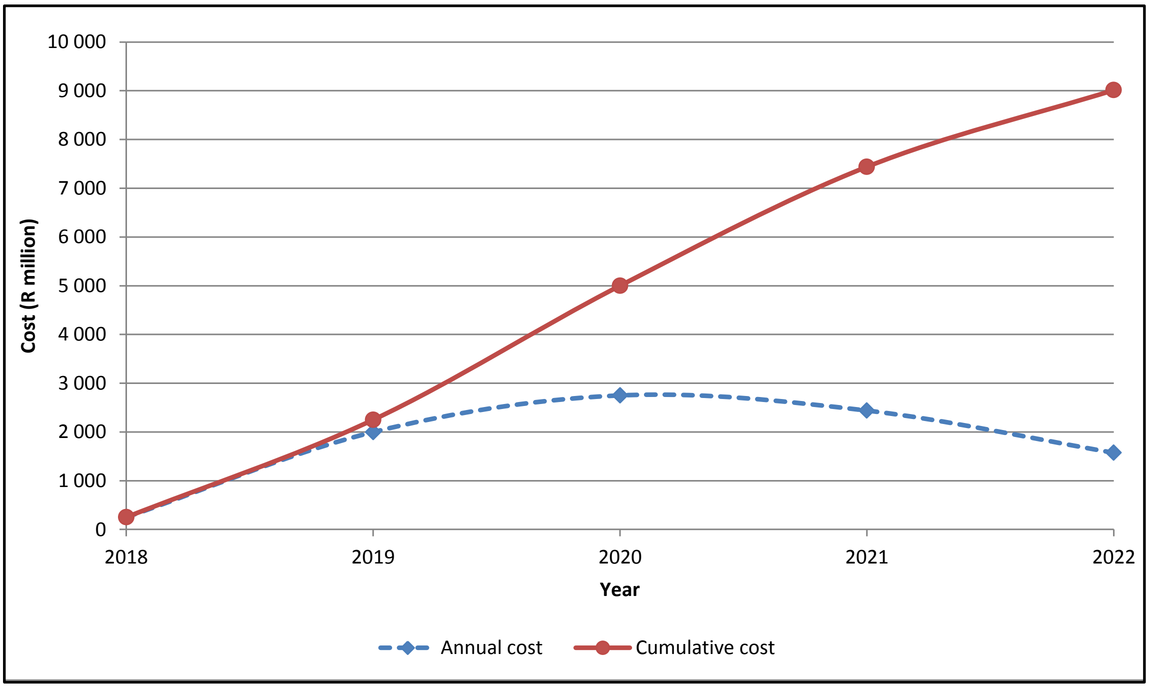

Figure 14.1: Construction cash flow forecast: activity cost only

14.5.2 Cash flow: Activity cost, including Ps&Gs and contingencies

Table 14.8 shows the cash flow forecast for construction, for the activity costs including Ps&Gs and contingencies.

Table 14.8: Construction cash flow forecast: Activity cost including Ps&Gs and contingencies

Item	Total capital cost (R million)	Capital cost per year (R million)				
		2018	2019	2020	2021	2022
		1	2	3	4	5
Raw water system						
Smithfield Dam	3 153	39	1 322	1 322	469	
Saddle embankment	497	39	229	229		
Main embankment	1 567		549	549	469	
All other components	1 088		544	544		
Langa Dam	686		47	319	319	
Main embankment	499		47	226	226	
All other components	187			93	93	
Tunnel	6 096	156	1 485	1 485	1 485	1 485
Raw water pipeline	571		78	246	246	
Access roads	363	181	181			
Gauging weirs	47		16	16	16	
Waste disposal sites	23	23				
Transmission lines	8	4	4			
Sub-total: Raw water system	**10 946**	**404**	**3 133**	**3 389**	**2 535**	**1 485**
Cumulative sub-total: Raw water system		**404**	**3 537**	**6 926**	**9 461**	**10 946**
Potable water system						
WTW	1 242			373	497	373
Potable water pipeline	1 786			536	715	536
Sub-total: Potable water system	**3 028**			**908**	**1 211**	**908**
Cumulative sub-total: Potable water system				**908**	**2 120**	**3 028**
Other components						
Baynesfield HPP	70				35	35
Smithfield HPP	60				30	30
Sub-total: Other components	**130**				**65**	**65**
Cumulative sub-total: Potable water system					**65**	**130**
Grand total	**14 104**	**404**	**3 133**	**4 297**	**3 811**	**2 458**
Cumulative grand total		**404**	**3 537**	**7 834**	**11 646**	**14 104**

Figure 14.2 depicts the above cash flow forecast for the total system (raw water, potable water and other components).

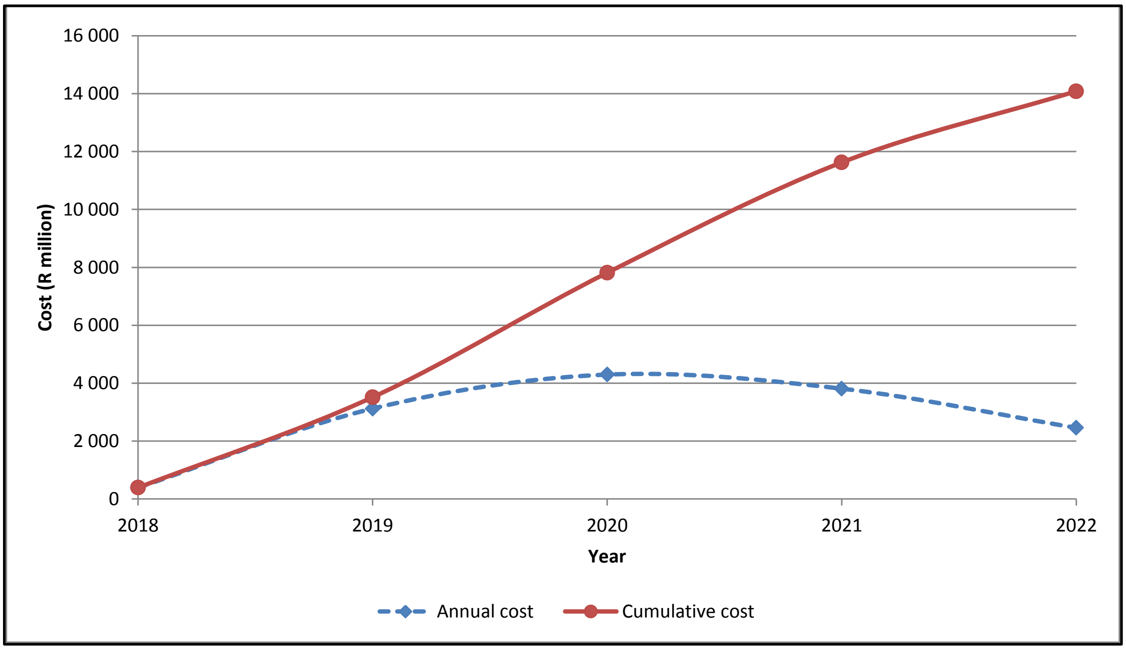

Figure 14.2: Construction cash flow forecast: Activity cost including Ps&Gs and contingencies

14.5.3 Cash flow: Activity cost, plus all additional costs

Table 14.9 shows the cash flow forecast for construction, for the activity costs plus all additional costs. These additional costs are Ps&Gs; professional fees; environmental, landscaping and social costs; land acquisition costs, contingencies and costs for implementing agents.

Table 14.9: Construction cash flow forecast: Activity cost plus all additional costs

Item	Total capital cost (R million)	Capital cost per year (R million)				
		2018	2019	2020	2021	2022
		1	2	3	4	5
Raw water system						
Smithfield Dam	3 455	43	1 449	1 449	514	0
Saddle embankment	545	43	251	251	0	0
Main embankment	1 718	0	602	602	514	0
All other components	1 193	0	596	596	0	0
Langa Dam	751	0	51	350	350	0
Main embankment	547	0	51	248	248	0
All other components	205	0	0	102	102	0
Tunnel	6 681	171	1 627	1 627	1 627	1 627
Raw water pipeline	625	0	86	270	270	0
Access roads	397	199	199	0	0	0
Gauging weirs	52	0	17	17	17	0
Waste disposal sites	26	26	0	0	0	0
Transmission lines	9	4	4	0	0	0
Sub-total: Raw water system	**12 805**	**472**	**3 666**	**3 964**	**2 966**	**1 737**
Cumulative sub-total: Raw water system		**472**	**4138**	**8102**	**11068**	**12805**
Potable water system						
WTW	1 467			440	587	440
Potable water pipeline	2 110			633	844	633
Sub-total: Potable water system	**3 591**	**0**	**0**	**1 077**	**1 436**	**1 077**
Cumulative sub-total: Potable water system		**0**	**0**	**1 077**	**2 514**	**3 591**
Other components						
Baynesfield HPP	77				38	38
Smithfield HPP	66				33	33
Sub-total: Other components	**143**	**0**	**0**	**0**	**71**	**71**
Cumulative sub-total: Potable water system		**0**	**0**	**0**	**71**	**143**
Grand total	**16 538**	**472**	**3 666**	**5 042**	**4 473**	**2 886**
Cumulative grand total		**472**	**4 138**	**9 180**	**13 653**	**16 538**

Figure 14.3 depicts the above cash flow forecast for the total system (raw water, potable water and other components).

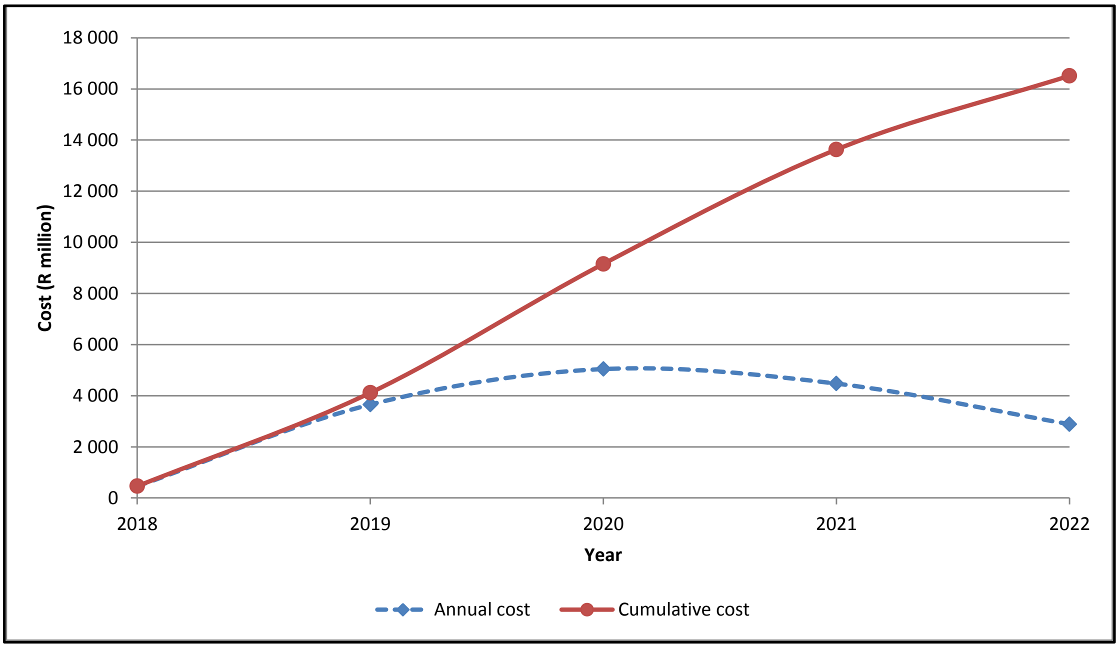

Figure 14.3: Construction cash flow forecast: Activity cost plus all additional costs

14.6 REFERENCES

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15 CONSTRUCTION PROGRAMME

A construction programme for this comprehensive multidisciplinary project, including the principle work items, with emphasis on the critical path activities, is shown in **Figure 15.1**. The duration of the various activities were based on realistic construction production rates, and the construction programme generated based on the following milestone dates:

- Commencement of construction date of September 2018;
- Commencement of Impoundment date of September 2022; and
- Commencement of water supply to Umgeni Water date of January 2023.

With the programme it was assumed that appropriate time has been allocated to complete pre-construction activities and preparations. These include:

- Tendering process and contract award;
- Obtaining of relevant approvals, permits and licenses;
- Financing; and
- Land acquisition.

Activity						Summer year 1						Winter year 1						Summer year 2						Winter year 2						Summer year 3						Winter year 3						Summer year 4						Winter year 4						Summer year 5		
Time (years)					2018				2019												2020												2021												2022											
Time (Months)					Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Time (Month numbers)	Unit	Quantity	Rate/month	Months	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52
PROJECT GENERAL ITEMS																																																								
Construction contract award				10 days	X																																																			
Smithfield Dam:																																																								
Commencement of supply																																																					x			
Mobilisation					Mo	Mo	Mo	Mo	Mo																																															
Site and Engineers' Offices - incl. services					Of	Of	Of	Of																																																
River crossing						RC	RC																																																	
Erect crusher and batching plant							EC	EC	EC																																															
Establish sand screening plant					SP	SP																																																		
Establish Site laboratory					SL	SL																																																		
Preparation of quarry and aggregates								QA	QA	QA	QA	QA																																												
De-mobilise																																																								Dem
Langa Dam:																																																								
Commencement of supply																																																					x			
Mobilisation					Mo	Mo	Mo	Mo	Mo																																															
Site and Ennigeers' Offices - incl. services					Of	Of	Of	Of																																																
River crossing						RC																																																		
Erect batching plant							EC	EC	EC																																															
Establish Site laboratory					SL	SL																																																		
Sourcing of quarry and aggregates								QA	QA	QA	QA	QA																																												
De-mobilise																																																								Dem
SMITHFIELD DAM																																																								
General																																																								
River diversion through two tunnels	-	-	-										Di	Di	Di	Di	Di	Di	Di	Di	Di	Di	Di	Di	Di	Di	Di	Di	Di	Di	Di	Di	Di	Di	Di																					
River diversion through one tunnel	-	-	-																																	Di	Di	Di	Di	Di	Di															
River diversion over Cofferdam 5	-	-	-																Di	Di	Di	Di	Di	Di																																
Commencement of impoundment																																															Im	Im	Im	Im	Im	Im	Im	Im	Im	Im
Upgrading of transmission lines																									De	De	De	De	De	De	De	De	De	De	De	De																				
Diversion Works																																																								
Excavate portals	m^3	300000	100000	3.0		Ex	Ex	Ex																																																
Construct Coffer Dam 1	m^3	880	1500	0.6		CD1																																																		
Construct Coffer Dam 2	m^3	2170	1500	1.4			CD2	CD2																																																
Tunnel 1 excavation	km	0.4	0.25	1.6			DB	DB																																																
Tunnel 2 excavation	km	0.4	0.25	1.6					DB	DB																																														
Tunnel 1 lining	km	0.4	0.1	4.0				Li	Li	Li	Li																																													
Tunnel 2 support	km	0.4	0.25	1.6						S	S																																													
Remove Coffer Dams 1 and 2												R1&2																																												
Coffer Dam 3	m^3	5460	3000	1.8								CD3	CD3																																											
Coffer Dam 4	m^3	10320	10000	1.0									CD4																																											
Coffer Dam 5	m^3	2970	900	3.3										CD5	CD5	CD5	CD5																																							
Remove Coffer Dam 3																								R 3																																
Coffer Dam 6	m^3	822165	150000	5.5																				CD6	CD6	CD6	CD6	CD6	CD6																											
Plug Tunnel 1																																				PT1																				
Plug Tunnel 2																																										PT2														
Reinstate access between tunnels																																											Ri													
Main Dam Embankment (ECRD)																																																								
Curtain grouting	m	23063	1320	17.5							G	G	G	G	G	G	G	G	G	G	G	G	G	G	G	G	G	G	G	G	G	G	G																							
Main dam core	m^3	855775	46200	18.5																Co	Co	Co	Co	Co	Co	Co	Co	Co	Co	Co	Co	Co	Co	Co	Co	Co	Co	Co	Co	Co	Co															
Main dam rockfill	m^3	3599640	220000	16.4																				RF	RF	RF	RF	RF	RF	RF	RF	RF	RF	RF	RF	RF	RF	RF	RF	RF	RF															
Saddle Dam (Zoned Earthfill)																																																								
Saddle Dam Curtain grouting	m^3	6650	1320	5.0			G	G	G	G	G																																													
Saddle Dam Core (zoned earthfill)	m^3	199900	46200	4.3					Co	Co	Co	Co	Co																																											
Saddle Dam Outer zones (zoned earthfill)	m^3	990500	46200	21.4						EF	EF	EF	EF	EF	EF	EF	EF	EF	EF	EF	EF	EF	EF	EF	EF	EF	EF	EF	EF	EF	EF																									
			2100																																																					
Spillway																																																								
Excavation	m^3	1687686	110000	15.3							E	E	E	E	E	E	E	E	E	E	E	E	E	E	E																															
Structures	m^3	50000	3300	15.2														C	C	C	C	C	C	C	C	C	C	C	C	C	C	C																								

Figure 15.1: Construction programme

Activity	Unit	Quantity	Rate/month	Months		Summer year 1						Winter year 1						Summer year 2						Winter year 2						Summer year 3						Winter year 3						Summer year 4						Winter year 4						Summer year 5		
Time (years)					2018				2019												2020												2021												2022											
Time (Months)					Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Time (Month numbers)					1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52
SMITHFIELD DAM																																																								
Outlets																																																								
Intake Tower of the Outlet works	m	77	2	38.5				IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT										
Intake to the uMkhomazi-uMlazi Tunnel	m	59	1.5	39.3						IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT							
Access Roads																																																								
Deviation of the R617	km	12.06	2	6.0			R	R	R	R	R	R																																												
Access road to Nonguqa	km	8.13	2	4.1	R	R	R	R	R																																															
Access road to tunnel inlet portal/intake to tunnel	km	0.23	2	0.1		R																																																		
Access road to Main Dam Embankment	km	1.56	2	0.8		R																																																		
Construction road	km	5.82	2	2.9		R	R	R																																																
Access road to dam outlet works (Intake Tower to	km	3	2	1.5	R	R																																																		
Finishing and rehabilitation of site																																																								
Rehabilitation of site																																																						Reh	Reh	
uMKHOMAZI-uMLAZI TUNNEL																																																								
General																																																								
Mobilization and erection of TBM's					Mo	Mo	Mo	Mo	Mo	Mo	Mo	Mo	Mo	Mo	Mo	Mo	Mo	Mo																																						
Excavation of portals	m^3	1500000	300000	5.0				EX	EX	EX	EX	EX																																												
Central access tunnel and mid section	km	2	0.25	8.0				DB	DB	DB	DB	DB	DB	DB	DB																																							
Other access adits	km	2	0.25	8.0											DB	DB	DB	DB	DB	DB	DB	DB																																
Tunnel: Chainage 14.8km to 0km	km	13.8	0.492	28.0															TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB										
Tunnel: Chainage 33.1km to 15.2 km	km	15.5	0.492	31.5															TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB	TB						
Shafts	m	270	40	6.8																			S2	S2	S2	S2		S3	S3	S3	S3	S3																								
Access roads																																																								
Access road to Ventilation/Air Shaft 2	km	0.07	2	1.0																R																																				
Access road to Ventilation/Air Shaft 3	km	0.068	2	1.0																	R																																			
Access road to centre adit entry	km	2.1	2	1.1	R	R																																																		
Waste disposal sites																																																								
Waste Disposal Site 1	m^3	600000	200000	3.0	LS	LS	LS																																																	
Waste Disposal Site 2	m^3	550000	200000	2.8	LS	LS	LS																																																	
uMLAZI-WATER TREATMENT PLANT PIPELINE																																																								
General																																																								
Excavation	m	5600	280	20.0																Ex	Ex	Ex	Ex	Ex	Ex	Ex	Ex	Ex	Ex	Ex	Ex	Ex	Ex	Ex	Ex	Ex	Ex	Ex	Ex																	
Pipe manufacturing and supply	m	5600	700	8.0								Man	Man	Man	Man	Man	Man	Man	Man																																					
Construction incl. testing of pipes	m	5600	280	20.0																	Con	Con	Con	Con	Con	Con	Con	Con	Con	Con	Con	Con	Con	Con	Con	Con	Con	Con	Con	Con																
LANGA BALANCING DAM																																																								
General																																																								
River diversion through bottom outlet conduits																																	Di	Di	Di	Di	Di	Di	Di	Di	Di	Di										
Commencement of impoundment																																											Im	Im	Im	Im	Im	Im	Im	Im		
Diversion Conduits																																																								
Excavation of bottom outlet portal	m^3	15000	10000	1.5													Ex	Ex																																						
Coffer Dams																																																								
Cofferdam 1	m^3	11600	2000	5.8								CD1	CD1	CD1	CD1	CD1	CD1																																							
Cofferdam 2	m^3	1700	1500	1.1																																CD2																				
Main Dam Embankment (CFRD)																																																								
Curtain grouting	m	8060	1000	8.1																	G	G	G	G	G	G											G	G																		
Rockfill, incl. protection layers	m^3	1628200	100000	16.3																				RF	RF	RF	RF	RF	RF	RF	RF	RF	RF	RF	RF	RF	RF	RF	RF	RF																
Concrete face	m^3	15920	1000	15.9																						CF	CF	CF	CF	CF	CF	CF	CF	CF	CF	CF	CF	CF	CF	CF	CF															
Spillway																																																								
Excavation	m^3	2530	110000	0.02										E																																										
Structures	m^3	365	3300	0.1											C																																									

Figure 15.1 – Construction programme (continued)

Activity	Unit	Quantity	Rate/month	Months		Summer year 1						Winter year 1						Summer year 2						Winter year 2						Summer year 3						Winter year 3						Summer year 4						Winter year 4						Summer year 5		
Time (years)					2018				2019												2020												2021												2022											
Time (Months)					Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Time (Month numbers)					1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52
LANGA BALANCING DAM																																																								
Outlets																																																								
Bottom Outlet	m	470	250	1.9															BO	BO																																				
Intake Structure	m	47	1.5	31.3														IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT	IT							
Access Roads																																																								
Access road to tunnel outlet portal and LBD (Optio	km	4.69	2	2.3		R	R	R																																															
Access road to WTW	km	2.37	2	1.2			R	R																																															
Finishing and rehabilitation of site																																																								
Rehabilitation of site				2.0																																														Reh	Reh	
GAUGING WEIRS																																																								
General																																																								
Weir DS of Impendle Dam	m^3	5870	1000	5.9								GW	GW	GW	GW	GW	GW																																						
Weir DS of Smithfield Dam	m^3	1600	500	3.2																					GW	GW	GW	GW																												
Weir DS of Smithfield Dam near Hella Hella	m^3	6260	1000	6.3																																GW	GW	GW	GW	GW	GW															
Access Roads																																																								
Access road to gauging weir 1	km	0.17	2	0.1							R																																													
Access road to gauging weir 2	km	2.165	2	1.1																			R																																	
Access road to gauging weir 3	km	2.516	2	1.3																																R																					

Figure 15.1 – Construction programme (continued)

15.1 CONSTRUCTION QUANTITIES AND PRODUCTION RATES

The construction quantities as well as the production rates used are indicated in **Table 15.1**. The production rates are associated with historic production rates of previous projects and based on a 22 day working month. The sources are listed below.

- Vaal Augmentation Planning Study (Consult 4, 1994);
- Ncwabeni Off-channel Storage Dam Feasibility Study: Module 1: Technical Study (BKS (Pty) Ltd, 2012);
- Lesotho Highlands Water Project; Consulting Services for Mohale Dam; Stage 1 Services; Tender Design and Preparation of Tender Documents (Mohale Consultants Group, 1998); and
- Mooi-Mgeni Transfer Scheme Phase 2; Consulting Services for Spring Grove Dam (BKS (Pty) Ltd, 2008).

Table 15.1: Production rates

No	Project component	Unit	Production rate			Time			Source or comments
			Volume	Rate/day	Rate/month	Days	Months	Years	
	SMITHFIELD DAM								
1	**General**								
1.1	Mobilisation	-	-	-	-	-	5	0.42	Normal estimate
1.2	Site and engineers' offices - incl. services	-	-	-	-	-	4	0.33	Based on Spring Grove Dam
1.3	River crossing	-	-	-	-	-	2	0.17	Normal estimate
1.4	Erect crusher and batching plant	-	-	-	-	-	3	0.25	Based on Spring Grove Dam
1.5	Establish sand screening plant	-	-	-	-	-	2	0.17	Based on tenders
1.6	Establish Site laboratory	-	-	-	-	-	2	0.17	Based on Spring Grove Dam
1.7	Preparation of quarry and aggregates	-	-	-	-	-	5	0.42	Based on contracts
1.8	De-mobilise	-	-	-	-	-	1.25	0.10	Based on Spring Grove Dam
1.9	Rehabilitation of site	-	-	-	-	-	2	0.17	Based on Spring Grove Dam
1.10	Deviation of Transmission lines	-	-	-	-	-	10	0.83	Eskom provided period of approximately 1 year
2	**Diversion tunnels**								
2.1	Excavate portals	m^3	300 000	4 545	100 000	66	3	0.25	Based on tenders
2.2	Tunnel 1 excavation	km	0.4	0.01	0.25	35	1.6	0.13	Mohale Dam average construction rate of 130 m^3/day
2.3	Tunnel 2 excavation	km	0.4	0.01	0.25	35	1.6	0.13	
2.4	Tunnel 1 lining	km	0.4	0.005	0.1	88	4	0.33	Mohale Dam construction rate of 9 m/36 hour
2.5	Tunnel 2 rock support	km	0.4	0.07	1.6	6	0.25	0.02	Rock support production rate assumed to be the same as that of the tunnel excavation production rate
2.6	Plug tunnels	No.	2	0.18	4	11	0.5	0.04	Estimate

No	Project component	Unit	Production rate			Time			Source or comments
			Volume	Rate/day	Rate/month	Days	Months	Years	
3	**Cofferdams**								
3.1	Cofferdam 1 (Earthfill)	m³	570	27	600	21	1.0	0.08	Production rate reduced from Ncwabeni Off-channel Storage Dam Feasibility Study due to space restriction
3.2	Cofferdam 2 (Earthfill)	m³	1 300	27	600	48	2.2	0.18	
3.3	Cofferdam 3 (Earthfill)	m³	3 360	76	1 680	44	2.0	0.17	
3.4	Cofferdam 4 (Rockfill)	m³	10 320	455	10 000	23	1.0	0.09	Based on Main Dam rockfill production rate (from Ncwabeni) but reduced due to finishing required
3.5	Cofferdam 5 (RCC)	m³	2 870	41	900	70	3.2	0.27	Production rate reduced from Ncwabeni Off-channel Storage Dam Feasibility Study due to limited space
3.6	Cofferdam 6 (Rockfill)	m³	82 460	4 545	100 000	18	0.8	0.07	Based on Main Dam rockfill production rate (from Ncwabeni) but reduced due finishing required
4	**Main Dam Embankment and Saddle Dam**								
4.1	Main dam curtain grouting	m	23 063	60	1 320	384	17.5	1.46	Based on Mohale Dam average construction rate of 60 m/day
4.2	Main dam rockfill	m³	3 599 640	10 000	220 000	360	16.4	1.36	Based on Ncwabeni Off-channel Storage Dam Feasibility Study production rates
4.3	Main dam core	m³	855 775	2 100	46 200	408	18.5	1.54	
4.4	Saddle dam curtain grouting	m	6 650	60	1 320	111	5.0	0.42	Based on Mohale Dam average construction rate of 60 m/day
4.5	Saddle dam earthfill core	m³	199 900	2 100	46 200	95	4.3	0.36	Based on Ncwabeni Off-channel Storage Dam Feasibility Study production rates
4.6	Saddle dam earthfill shell	m³	990 500	2 100	46 200	472	21.4	1.79	
5	**Spillway**								
5.1	Excavation	m³	1 687 686	5 000	110 000	338	15.3	1.28	Based on Ncwabeni Off-channel Storage Dam Feasibility Study production rates
5.2	Structures	m³	50 000	150	3 300	333	15.2	1.26	
6	**Outlets**								
6.1	Intake tower of outlet works	m	77	-	2	-	38.5	3.21	37 m high Spring Grove Dam intake tower and other outlet works infrastructure were completed within 2 years, thus a production rate was assumed per m height of each intake structure
6.2	Intake to tunnel	m	59	-	1.5	-	39.3	3.28	

No	Project component	Unit	Production rate			Time			Source or comments
			Volume	Rate/day	Rate/month	Days	Months	Years	
7	**Roads**								
7.1	Deviation of the R617	km	12.06	0.091	2	133	6.0	0.5	Production rates are only an estimate and need to be verified during the Detail Design Phase
7.2	Access road to Nonguqa	km	8.13	0.091	2	89	4.1	0.3	
7.3	Access road to tunnel inlet portal/intake to tunnel	km	0.23	0.091	2	3	0.1	0.0	
7.4	Access road to main dam embankment	km	1.56	0.091	2	17	0.8	0.1	
7.5	Construction road	km	5.82	0.091	2	64	2.9	0.2	
7.6	Access road to dam outlet works (intake tower to dam)	km	3	0.091	2	33	1.5	0.1	
	uMKHOMAZI-uMLAZA TUNNEL								
1	**General**								
1.1	Mobilization and erection of TBMs	No.	2		0.15	-	13.3	1.11	Based on Mohale Dam construction production rates: 9-12 lead time and 3-6 weeks for assembly of machines
1.2	Excavation of portals	m³	1 500 000	13 636	300 000	110	5.0	0.42	Based on tenders
1.3	Central access tunnel and mid-section	km	2	0.01	0.25	176	8.0	0.67	Mohale Dam average construction rate of 130m3/day (Drill and blast)
1.4	Other access adits	km	2	0.01	0.25	176	8.0	0.67	
1.5	Tunnel: Chainage 14.8 km to 0 km	km	13.8	0.02	0.492	617	28.0	2.34	Based on Mohale Dam average construction rate of 130m/week
1.6	Tunnel: Chainage 33.1 km to 15.2 km	km	15.5	0.02	0.492	693	31.5	2.63	
1.7	Shafts	m	270	2	40	149	6.8	0.56	Based on other contracts
2	**Roads**								
2.1	Access road to Ventilation Shaft 2	km	0.07	-	2	-	0.04	0.003	Production rates are only an estimate and need to be verified during the Detail Design Phase.
2.2	Access road to Ventilation Shaft 3	km	0.068	-	2	-	0.03	0.003	
2.3	Access road to centre adit entry	km	2.1	-	2	-	1.1	0.1	
3	**Waste disposal sites**								
3.1	Waste Disposal Site 1	m³	600 000	-	200 000	-	3.0	0.3	Must be completed before majority of spoil material is generated. Need to be revised during the Detail Design Phase.
3.2	Waste Disposal Site 2	m³	550 000	-	200 000	-	2.8	0.2	

No	Project component	Unit	Production rate			Time			Source or comments
			Volume	Rate/day	Rate/month	Days	Months	Years	
	BAYNESFIELD RAW WATER PIPELINE								
1	**General**								
1.1	Excavation	m	5 600	-	280	-	20.0	1.7	Based on Spring Grove Dam production rates
1.2	Pipe manufacturing and supply	m	5 600	-	700	-	8.0	0.7	
1.3	Construction	m	5 600	-	280	-	20.0	1.7	
	LANGA DAM								
1	**General**								
1.1	Mobilisation	-	-	-	-	-	5	0.42	Normal Estimate
1.2	Site and engineers' offices - incl. services	-	-	-	-	-	4	0.33	Based on Spring Grove Dam
1.3	River crossing	-	-	-	-	-	1	0.08	Normal Estimate
1.4	Erect batching plant	-	-	-	-	-	1.5	0.13	Based on Spring Grove Dam, however no need for crusher to be erected.
1.5	Establish site laboratory	-	-	-	-	-	2	0.17	Based on Spring Grove Dam
1.6	Sourcing of quarry and aggregates	-	-	-	-	-	1	0.08	Based on other contracts
1.7	Rehabilitation of site	-	-	-	-	-	2	0.17	Based on Spring Grove Dam
1.8	De-mobilise	-	-	-	-	-	1.25	0.10	Based on Spring Grove Dam
2	**Diversion Conduits**								
2.1	Excavation of bottom outlet portal	m^3	15 000	-	10 000	-	1.5	0.13	Based on tenders
2.2	Bottom outlet conduits	m	470	-	250	-	1.9	0.16	Based on Spring Grove Dam production rates
3	**Cofferdams**								
3.1	Cofferdam 1	m^3	11 600	-	2 000	-	5.8	0.48	Production rate reduced from Ncwabeni Off-channel Storage Dam Feasibility Study due to limited space.
3.2	Cofferdam 2	m^3	1 700	-	1 500	-	1.1	0.09	

No	Project component	Unit	Production rate			Time			Source or comments
			Volume	Rate/day	Rate/month	Days	Months	Years	
4	**Main Dam Embankment**								
4.1	Curtain grouting	m	8 060	45	1 000	177	8.1	0.67	Based on Mohale Dam average construction rate of 60 m/day.
4.2	Main dam rockfill	m³	1 628 200	4 545	100 000	358	16.3	1.36	Based on Ncwabeni Off-channel Storage Dam Feasibility Study production rates.
4.3	Main dam concrete face	m³	15 920	45	1 000	350	15.9	1.33	Based on Mohale Dam face slab production rate.
5	**Spillway**								
5.1	Excavation	m³	2 530	5 000	110 000	0.5	-	-	Based on Ncwabeni Of-channel Storage Dam Feasibility Study production rates. Very small spillway required.
5.2	Structures	m³	365	150	3 300	2.4	-	-	
6	**Outlets**								
6.1	Intake structure	m	47	-	1.5	-	31.3	2.6	See comment on Smithfield Dam outlets.
7	**Roads**								
7.1	Access road to tunnel outlet portal and Langa Dam (Option 2)	km	4.69	-	2	-	2.3	0.2	Production rates are only an estimate and need to be verified during the Detail Design Phase.
7.2	Access road to WTW	km	2.37	-	2	-	1.2	0.1	
	GAUGING WEIRS								
1	**Weirs**								
1.1	Weir U/S of Smithfield Dam	m³	5 870	-	1 000	-	6	0.5	Approximately six months allocated to each weir, based on Spring Grove Dam. Includes excavation and construction of coffer dams.
1.2	Weir D/S of Smithfield Dam	m³	1 600	-	500	-	6	0.5	
1.3	Weir near EWR/IFR2	m³	6 260	-	1 000	-	6	0.5	
2	**Roads**								
2.1	Access road to gauging weir 1	km	0.17	-	2	-	0.1	0.0	Production rates are only an estimate and need to be verified during the Detail Design Phase.
2.2	Access road to gauging weir 2	km	2.165	-	2	-	1.1	0.1	
2.3	Access road to gauging weir 3	km	2.516	-	2	-	1.3	0.1	

15.2 Critical Path

From the construction programme it is clear that the ***critical path follows the uMkhomazi – uMlaza Tunnel construction preparations and activities***. These include:

- Procurement and mobilisation of the TBM;
- Provision of river crossing;
- Erection of the crusher and batching plant;
- Drilling and blasting of the central access tunnel at mid length of the tunnel;
- Drilling and blasting of other access adits; and
- Boring and lining of the uMkhomazi – uMlaza Tunnel from chainage 33.1 km to chainage 15.2 km.

All the other facilities can be completed within this critical period. However, the completion of other construction activities is also crucial even though not on the critical path. These activities include:

- Construction of access roads;
- Excavation and lining of the Smithfield Dam river diversion tunnel 2;
- Construction of Smithfield Dam RCC Cofferdam 5;
- Construction of Smithfield Dam Rockfill Cofferdam 6; and
- Construction of a large portion of the Smithfield Dam saddle embankment.

Roads providing access to hard-to-reach construction sites have to be completed before the associated construction activity can commence. The production rates for the roads are only estimates as the exact production rate of each road highly depends on the required amount of cut and fill. Hence production rates for the construction of all roads needs to be refined during the detail design phase.

The excavation and lining of Smithfield Dam river diversion tunnel 2 and the rock support of Smithfield Dam river diversion tunnel 1 has to be completed by the end of March 2019 to start diverting water through the tunnels.

Smithfield Dam Cofferdam 5 must be completed before the summer rain season of 2019 to avoid possible floods to enter the construction area and consequently cause damage and delay construction. The same applies for Cofferdam 6 and the summer rain season of 2020.

A large portion of the Smithfield Dam saddle embankment has to be completed before the construction of the main embankment can commence as the dolerite required for the shell of the main embankment is overlain by the shale to be used for the saddle embankment shell.

15.3 SUMMARY AND RECOMMENDATIONS

To adhere to the dates given in **Section 15**, it is of utmost importance to stay on schedule with the construction programme, especially the activities associated with the critical path as well as the other activities deemed important.

The construction programme needs to be reviewed and adjusted accordingly after more accurate quantities and production rates have been established during the detail design phase.

It is recommended that all must be done to commence construction in 2017 to ensure that delays can be mitigated.

15.4 REFERENCES

BKS (Pty) Ltd, 2008. *Mooi-Mgeni Transfer Scheme Phase 2; Consulting Services for Spring Grove Dam (Contract 04-041).* South Africa: Trans-Caledon Tunneling Authority (TCTA).

BKS (Pty) Ltd, 2012. *Ncwabeni Off-channel Storage Dam Feasibility Study: Module 1: Technical Study; Supporting Report 3: Design and Cost Estimates (Report no. P WMA 11/T52/00/3311/3).* Pretoria, South Africa: Department of Water Affairs (DWA).

Consult 4, 1994. *Vaal Augmentation Planning Study; Guidelines for the Preliminary Sizing, Costing and Engineering Economic Evaluation of Planning Options (P C000/00/14394).* Pretoria, South Africa: Department of Water Affairs (DWA).

Mohale Consultants Group, 1998. *Lesotho Highlands Water Project; Consulting Services for Mohale Dam; Stage 1 Services; Tender Design and Preparation of Tender Documents; Interim Design Report (Report no. 1017-1.7-02).* Maseru, Lesotho: Kingdom of Lesotho Highlands Development Authority.

16 CONCLUSIONS AND RECOMMENDATIONS

The conceptual design of the selected scheme of the raw water component of the uMkhomazi Water Project is described in this report. This scheme comprises the following main components:

- Smithfield Dam, with a full supply level of 930 masl and consisting of an earth core rockfill main embankment and a zoned earthfill saddle embankment;
- The uMkhomazi – uMlaza Tunnel, with a finished internal diameter of 3.5 m and a length of 32.5 km;
- The Baynesfield Raw Water Pipeline, with two sections of 2.6 and 1.6 m diameters and 5.2 and 1.3 km lengths, respectively; and
- Langa Dam, a concrete faced rockfill dam with a full supply level of 923 masl.

Specific recommendations for the different components have been made at the end of each section of this report.

The main conclusions of the design are as follows:

- The construction programme is scheduled for implementation from 2018 to 2022. It is important that construction begins at 2018 at the latest.
- The total cost estimate for construction of the raw water system is R 12 946 million, excluding VAT (R 14 758 million, including VAT). This amount includes P&G costs, professional fees and contingencies. Financing costs are not included in this cost. In order for construction to begin in 2018, the detailed tender design must commence in 2016.
- The land that is required for construction of the scheme is to be acquired in time for construction.